SECTION VII

IMPACT OF URBAN WATER POLLUTION CONTROL ON RECEIVING WATER QUALITY

PROBLEM DEFINITION

Unquestionably, the nationwide commitment toward control of water pollution and the efficient use of available water resources has led to the need for comprehensive and integrated river basin planning. An average annual precipitation of 30 inches (762 mm) constitutes the basic source of water supply for the conterminous US, an area of 3,022,387 square miles (7,827,982 sq kms).[1] This amount corresponds to approximately 4,300 billion gallons per day (16.3×10^9 cu m/day). Surface runoff, interflow and groundwater flow result in an average annual streamflow of about 1,200 billion gallons per day (4.54×10^9 cu m/day).[2] This is a simplified measure of the total available freshwater supply. Estimates of water use in the United States for 1970 indicate that 370 billion gallons per day (1.40×10^9 cu m/day) were withdrawn to satisfy off-channel demand. Of the total withdrawn, 87 billion gallons per day (0.33×10^9 cu m/day) were estimated to be consumed -- that is, water made unavailable for further possible withdrawal by natural evaporation, incorporation into crops and manufactured products, and other causes. For the 5-year period from 1965 to 1970, withdrawals increased by 19 percent and consumptive water use increased 12 percent. It is not difficult to visualize that if withdrawals in 1970 were already 31 percent of the total available supply, and consumption was 7 percent, strict water management practices will be absolutely necessary to cope with an increasing water demand upon a fixed water resource. The quality of a scarce freshwater supply will have an added significance for obvious reasons.

The water quality cycle is a dynamic system existing within each phase of the hydrologic cycle. The amount of pollution entering or leaving a water body is determined by the quantity of flow and concentration of pollutants in each of the hydrologic components of the physical system. The retention of pollutants in the water body is not solely a function of the quantity and quality of hydraulic flows, since it also depends upon the location of the pollutants within the water body. The pollutants exist in the water, bottom sediments, and the aquatic life. When area sources such as bottom sediments are the greatest pollutant contributors, it may be necessary to consider treatment of the water body itself; for example, stream aeration. Thus, one of the initial steps in the planning process is water quality problem identification.

Urban areas represent the centers of most intense human activity. Urban land use is small within a river basin, as evidenced in Table VII-1, Land Use in the United States.[3] Point discharges resulting from commercial, industrial, and residential wastes generally enter the receiving stream within relatively short distances of each other, and in some cases all such wastes are processed by municipal facilities and discharged to the water body at one location. Thus, these continuous waste products are concentrated within a relatively small volume of the receiving water. Intermittent precipitation falling on urban areas becomes contaminated as it enters and passes through or within the manmade environment. The first quality degradation occurs when the rainwater comes into contact with pollutants in the air. Next, the surface runoff passes over ground and building surfaces, carrying suspended sediment from erosion sites and dissolving other impurities. Finally, the stormwater runoff comes into contact with: 1) solid residues deposited from earlier storms throughout the conveyance system and appurtenances; and 2) dry weather flow (DWF) in combined sewer systems. This storm runoff is well mixed with sanitary sewage under conditions of turbulent flow in a combined sewer system, and it eventually discharges to the receiving stream. The degradation undergone by urban stormwater passing through the surface runoff phase of the hydrologic cycle can be several orders of magnitude greater than that experienced by rainwater during the precipitation phase.[4] The pollutants either decompose (nonconservative), accumulate (benthic deposits), or are carried further downstream (conservative, suspended and dissolved matter).

The essence of a rational water quality and quantity management program is the decision making process. The high cost of pollution control facilities, in terms of both energy utilization and financial burden, obligates the planning agency to select the optimal strategy for areawide wastewater management. Such a process must focus on a systematic procedure that identifies and defines: 1) the cause/effect relationships of the physical environment; 2) the economic realities of control alternatives; and 3) the benefits to be derived from implementation of these controls. A preliminary analysis that provides an approximation of system responses to proposed treatment measures should aid the selection of the best strategy for restoration of water bodies to accepted water quality standards. Such an analysis must never be interpreted as other than a guide to be tempered by professional judgment. The mathematical models applied need not incorporate all phenomena but rather should be relevant to the problem under consideration. The problem of specific interest is to assess the separate and combined effects of the major urban sources of water pollution upon the quality of the receiving waters. Oxygen concentration is considered the key to the quality of natural water bodies, although it certainly is not the only water quality indicator.[5] Thus, the relative impact of these wastewater sources is appraised by their effect on the dissolved oxygen concentrations downstream from the urban area.

Table VII-1. LAND USE IN THE UNITED STATES (US Dept. of Agr. Econ. Rept. 149, 1968)[3]

	Acres	Hectares	Percentage
Privately-owned Land	1,326,642,000	536,892,000	59
Nonfederal Public Land	61,307,000	24,811,000	3
Indian Land	50,400,000	20,397,000	2
Small Water Areas	7,099,000	2,873,000	< 1
Urban and Built-up Areas	60,993,000	24,684,000	3
Federal Land	759,499,000	307,369,000	33
Total Areas, 50 States	2,265,940,000	917,026,000	100

It is of further interest to distinguish clearly between the two types of urban stormwater runoff, separate sewer flow and combined sewer overflow, and their relative pollutional impacts.

In essence, the mathematical model must be responsive to the land use, hydrology, and climatology of the drainage area while performing the following functions:

- generate stormwater runoff pollutant loads and dry-weather sanitary flow pollutant loads;
- simulate the pollutant removal efficiency of various treatment schemes;
- simulate the conveyance system, including mixing in combined sewers of wet- and dry-weather pollutants;
- mix the various pollutant inflow combinations with pollutants already in the receiving water (from upstream sources); and
- predict the oxygen balance of the polluted waters downstream from the waste sources,

subject to the constraints imposed by the quantity and quality of the data base. Each pollution control alternative is weighed on the basis of cost-effectiveness: the annual total cost and the water quality improvement obtained. Within the framework of mathematical modeling is the very important task of verification. An application of the model to Des Moines, Iowa, is presented. Computed values for dissolved oxygen are compared with field measurements.

METHODOLOGY

Characterization of Wastewater Discharges and Polluted Waters

To fully characterize the strength of wastewater discharges and the quality of polluted waters, large and diverse numbers of chemical, physical, biological, and bacteriological methods of analysis have been developed. The most common parameters measured are listed in Table VII-2, Pollution and Contamination Indices. The bacteriological procedures are designed to detect potential health hazards from contamination of the water with human or animal feces. The sampling technique, frequency of sampling, and method of preservation should be tailored to the indicators chosen for measurement. Field observations are extremely valuable for verification of mathematical models. Greater precision may be obtained in the laboratory, but if the sample is unrepresentative, greater accuracy will be achieved by *in situ* procedures.

Table VII-2. POLLUTION AND CONTAMINATION INDICES

Physical Parameters:
Temperature
Turbidity
Color
Chemical Parameters:
Oxygen Demand
Biochemical Oxygen Demand (BOD)
Chemical Oxygen Demand (COD)
Total Organic Carbon (TOC)
Nitrogen Compounds
Organic
Nitrite
Nitrate
Phosphorus Compounds
Ortho Phosphorus
Poly Phosphates
Total Solids
Dissolved
Suspended
Volatile and Fixed
Settleable
Chlorides
Sulfates
pH
Alkalinity
Hardness
Heavy Metals
Lead
Copper
Zinc
Chromium
Mercury
Biological Parameters:
Plankton
Periphyton
Macrophyton
Macroinvertebrates
Fish Bioassays
Bacteriological Parameters:
Total Coliform Count
Fecal Coliform
Fecal Streptococci
Total Plate Count

The wastewater constituents that affect the distribution of dissolved oxygen (DO) in a natural water system are well documented. The oxygen demand of sewage, sewage treatment plant effluent, polluted stormwater runoff or industrial wastes is exerted by three types of materials:[5,6]

1. carbonaceous organic matter oxidized by heterotrophic bacteria for energy and cell synthesis;

2. organic nitrogen compounds hydrolyzed into ammonia-nitrogen (NH_3-N), then oxidized by autotrophic bacteria (_Nitrosomonas europaea_) to nitrite-nitrogen (NO_2-N), further oxidized by _Nitrobacter winogradskyi_ to nitrate-nitrogen (NO_3-N); and

3. certain chemical reducing compounds (ferrous iron, sulfite, and sulfide) which will react with molecularly dissolved oxygen.

This oxygen demand is the response of aquatic biota to an adequate food supply and is commonly referred to as the biochemical oxygen demand (BOD). The laboratory BOD technique is an empirical bioassay-type procedure: the DO consumed by microbial life in an incubated bottle is measured with respect to time at a specified temperature. The actual environmental conditions of temperature changes, biological population, water movement, sunlight, and zones of aerobic and anaerobic processes cannot be faithfully reproduced in the laboratory. Thus, the "bottled" system, on a kinetic comparison, is completely accurate in representing itself but may be relatively unreliable as a representation of the source from which the sample was taken. The basic assumption that consumption of DO is an absolute and complete parameter of biological decomposition in the BOD bottle constitutes a simplification of complex interactions.

Other laboratory methods have been developed to measure the oxygen demand exerted by organic matter. The chemical oxygen demand (COD) and total organic carbon (TOC) tests are more precise _chemical_ methods, but the analytical results are not accurate if the organic material measured is not equivalent to the organic matter actually being utilized by the microorganisms in the stream. Much can be done to improve the accuracy of the BOD test by using dilution water from the receiving stream, thus introducing a natural "seed" of diverse organisms into the bottle system. With all of its limitations, the BOD procedure is still considered to be the best method for evaluating the effect of waste inputs on the oxygen balance of a stream.[7]

Model Overview

Data for the study area are used to simulate the hypothetical response of the receiving water to the separate and combined effects of BOD waste inputs from: 1) upstream sources, 2) dry-weather urban sources, and 3) wet-weather urban sources. The general concept is illustrated in Figure VII-1, Simplified Configuration of Mixed Waste Inputs to Receiving Water. The urban community served by a separate sewer system will convey stormwater runoff and municipal sewage through conduits which are not connected together. The BOD concentration of the storm sewer runoff is mixed with the dry-weather flow (DWF) and accumulated sewer solids. An interceptor carries the sanitary design flow to the municipal sewage treatment plant. The combined sewer overflow is either given treatment or allowed to discharge directly to the receiving water. Since complete mixing is assumed, the BOD concentrations of the combined sewer overflow (Q_c) and the flow (DWFCMB) intercepted for treatment by the DWF facility are identical. Any degree of treatment desired may be imposed at both the DWF and the WWF treatment plants. The concentration of the combined BOD inputs in the receiving water is given by:

$$BOD_m = \frac{BOD_u Q_u + BOD_d Q_d + BOD_w Q_w}{Q_u + Q_d + Q_w} \qquad \text{(VII-1)}$$

where BOD_m = mixed BOD concentration in receiving water, mg/1,

BOD_u = mixed BOD concentration from sources upstream of urban area, mg/1,

BOD_d = BOD concentration of dry-weather flow treatment plant effluent, mg/1,

BOD_w = BOD concentration of wet-weather flow treatment facility effluent, mg/1,

Q_u = upstream flow, cfs,

Q_d = DWF treated effluent, cfs, and

Q_w = WWF treated effluent, cfs.

The technique for calculation of the quantity and quality of stormwater and combined sewer overflows is discussed in further detail subsequently. The BOD concentrations of the DWF and WWF treated effluents are given by:

$$BOD_d = \frac{[BOD_f \cdot DWFSEP + BOD_c \cdot DWFCMB](1-R_d)}{DWFSEP + DWFCMB} \qquad \text{(VII-2)}$$

$$BOD_w = \frac{[BOD_s \cdot Q_s + BOD_c \cdot Q_c](1-R_w)}{Q_s + Q_c} \qquad \text{(VII-3)}$$

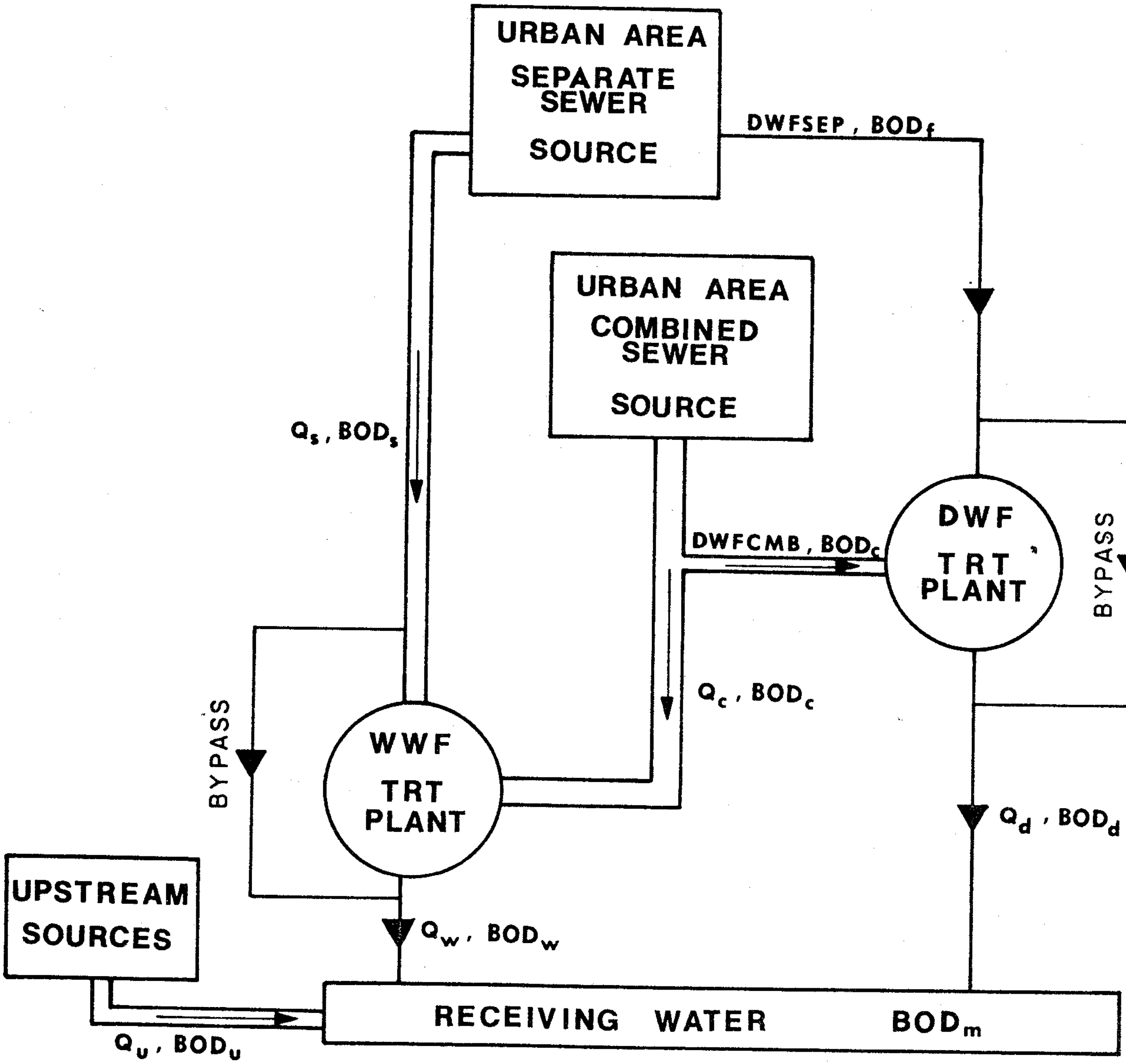

Figure VII-1. Simplified Configuration of Mixed Waste Inputs to Receiving Water where BOD_f = BOD concentration of municipal sewage, mg/l,

BOD_c = mixed BOD concentration in the combined sewer, mg/l,

BOD_s = BOD concentration of urban stormwater runoff, mg/l,

DWFSEP = DWF contribution from separate sewer area, cfs,

DWFCMB = DWF contribution from combined sewer area, cfs,

Q_s = urban runoff carried by the separate storm sewer, cfs,

Q_c = combined sewer flow , cfs,

R_d = fraction removal of BOD achieved by the DWF treatment facility, and

R_w = fraction removal of BOD achieved by the WWF treatment facility.

The initial conditions of BOD in the river are defined by equation VII-1, and the hypothetical impact on the oxygen balance of the receiving stream is estimated by using simplified mathematical modeling approaches. The total hours of runoff-producing rainfall throughout the year are separated into storm events by defining a minimum interevent time. The procedure is discussed in detail subsequently. For a given storm event, the runoff and pollutant loads are summed and the critical DO deficit is estimated as a function of several stream parameters: temperature, flow, oxygen concentration, deoxygenation and reaeration rates, and BOD concentrations. The minimum DO is calculated subsequently and a frequency analysis is performed. Stream velocity is computed as a function of the discharge and the time and distance to each critical deficit point are obtained for each event.

The options used for the simulations include:

1. five inflow combinations:

   a. river flow + DWF
   b. river flow + DWF + separate flow
   c. river flow + DWF + combined flow
   d. river flow + separate flow + combined flow
   e. river flow + separate flow + combined flow + DWF,

2. four DWF treatment rates (variable),

3. three WWF treatment rates (variable), and

4. three fractions of measured upstream flow

may be investigated.

Item 4 is included as a model option to investigate whether the relative impact of urban stormwater runoff is most significant in the upstream portions of river basins. This effect may be simulated by simply reducing the upstream flow to any desired fraction of its actual measured value. Thus, discharge into a dry river bed may be studied.

Technique for Calculation of Urban Runoff Quantity and Quality

This section briefly describes the methods used to generate storm runoff and pollutant concentrations. The Hydrologic Engineering Center model, STORM[8], is utilized to obtain hydrographs and pollutographs for Des Moines for the year 1968 on an hourly time step.

Urban Runoff Quantity --

As described in Section VI, STORM computes urban runoff as a function of land use and rainfall/snowmelt losses:

$$AR_u = CR_u(P_u - f_u) \qquad \text{(VII-4)}$$

where AR_u = urban area runoff, in./hr,

CR_u = composite runoff coefficient dependent on urban land use,

P_u = hourly rainfall/snowmelt in inches over the urban area, and

f_u = available urban depression storage, in.

A maximum depression storage of a hundredth of an inch (0.25 mm) is assumed for Des Moines, Iowa. The hourly urban runoff values, expressed in cfs, are saved in a file for later recall by the simplified mathematical model.

Urban Runoff Quality --

The basic water quality parameters modeled by STORM are suspended and settleable solids, BOD, total nitrogen (N), and total phosphate (PO_4). It is important to emphasize that the BOD values are expressed in terms of the standard BOD_5 test: incubation at a temperature of 20°C for 5 days. These values represent most of the <u>carbonaceous</u> oxygen demand exerted by organic matter present in the urban runoff, and include the BOD contribution from suspended and settleable solids. The BOD loading rates generated by STORM are based on land use and other factors such as number of dry days without runoff since the last storm and the street sweeping intervals.

However, these loading rates were calibrated against yearly averages and single storm values obtained from a detailed study of Des Moines by Davis and Borchardt.[9] The hourly BOD_5 pollutographs, in pounds per hour and mg/l, are also saved in a digital computer file for later recall by the receiving water simulation.

Definition of an Event

As stated previously, rainfall input to STORM is prepared as a sequence of consecutive hourly values (including zeros for no measurable precipitation). These inputs are used by STORM to generate the corresponding series of hourly urban runoff. The basic approach to define a wet-weather event is to analyze the hydrologic time series and establish the minimum number of consecutive dry-weather hours (DWH) that separates independent storm events. The independence of these storm events is not strictly climatological and is discussed later. The dry-weather hours refer to periods during which no runoff was produced. Thus, depression storage and evaporation rates must be satisfied before any runoff is generated by STORM.

Two techniques are used to analyze the hydrologic time series: 1) an analytic approach, autocorrelation, and 2) a graphic procedure. The precipitation time series is presented in Figure VII-2, Point Rainfall for Des Moines, Iowa. The abscissa represents the 10-month period, in hours, from March 1 to December 30, 1968. An examination of the rainfall record provides considerable insight as to the storm groupings, their intensity and duration, and frequency of occurrence. The broken line on the abscissa indicates dry-weather periods at least 9 hours in length. Figure VII-2 provides necessary information to apply both techniques and define a minimum interevent time.

For hydrologic processes, it is practical to estimate the autocorrelation coefficients by an open-series approach:[10,11]

$$r_I(k) = \frac{\sum_{i=1}^{n-k} x_i x_{i+k} - \frac{1}{n-k}\left[\sum_{i=1}^{n-k} x_i\right]\left[\sum_{i=k+1}^{n} x_i\right]}{\left[\sum_{i=1}^{n-k} x_i^2 - \frac{1}{n-k}\left(\sum_{i=1}^{n-k} x_i\right)^2\right]^{0.5}\left[\sum_{i=k+1}^{n} x_i^2 - \frac{1}{n-k}\left(\sum_{i=k+1}^{n} x_i\right)^2\right]^{0.5}} \qquad \text{(VII-5)}$$

where $r_I(k)$ = sample estimate of lag-k autocorrelation coefficient for hydrologic process I,

x_i = discrete data series (observations) of hydrologic process I, for $i = 1,2,\ldots,n$,

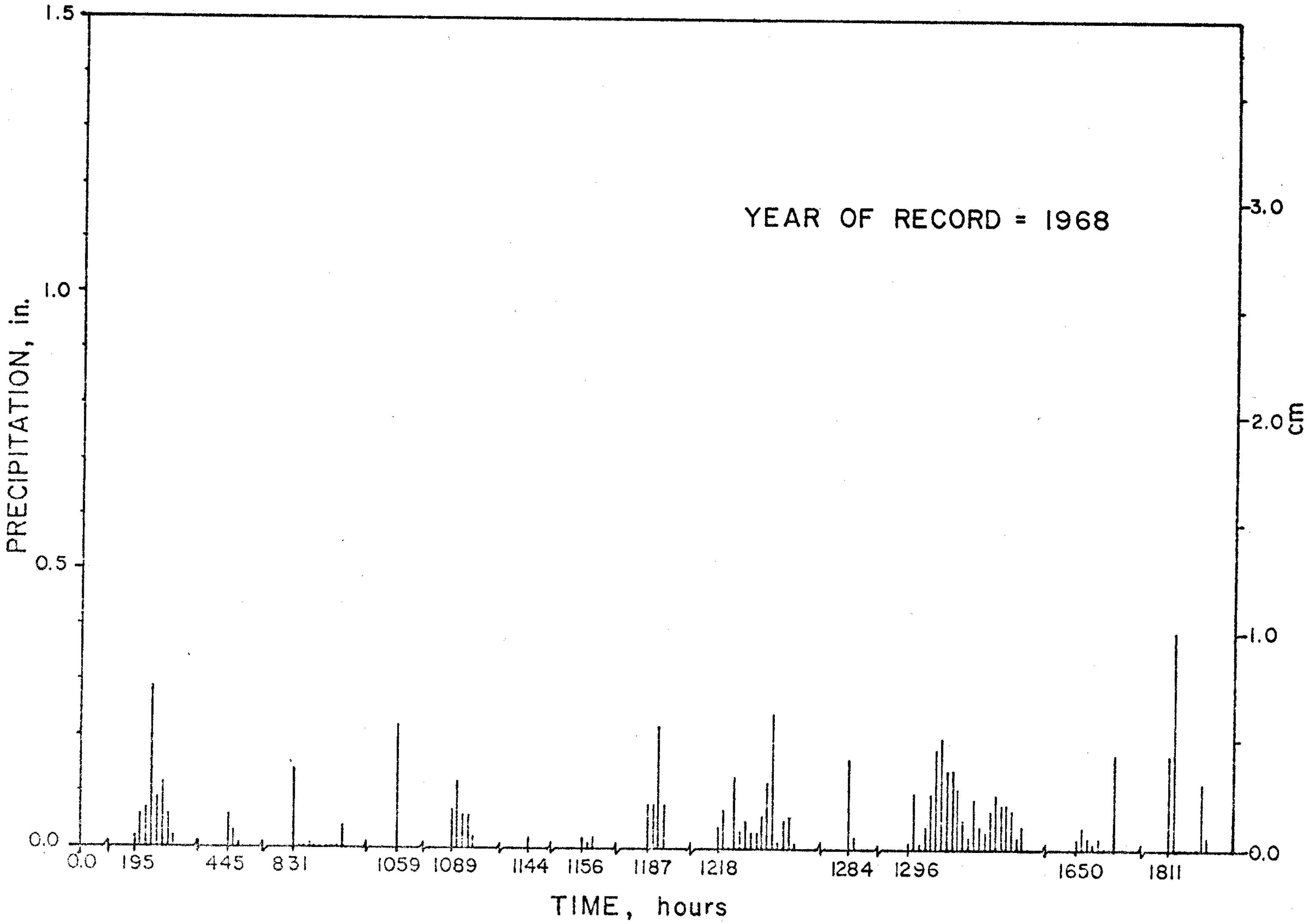

Figure VII-2. Point Rainfall for Des Moines, Iowa

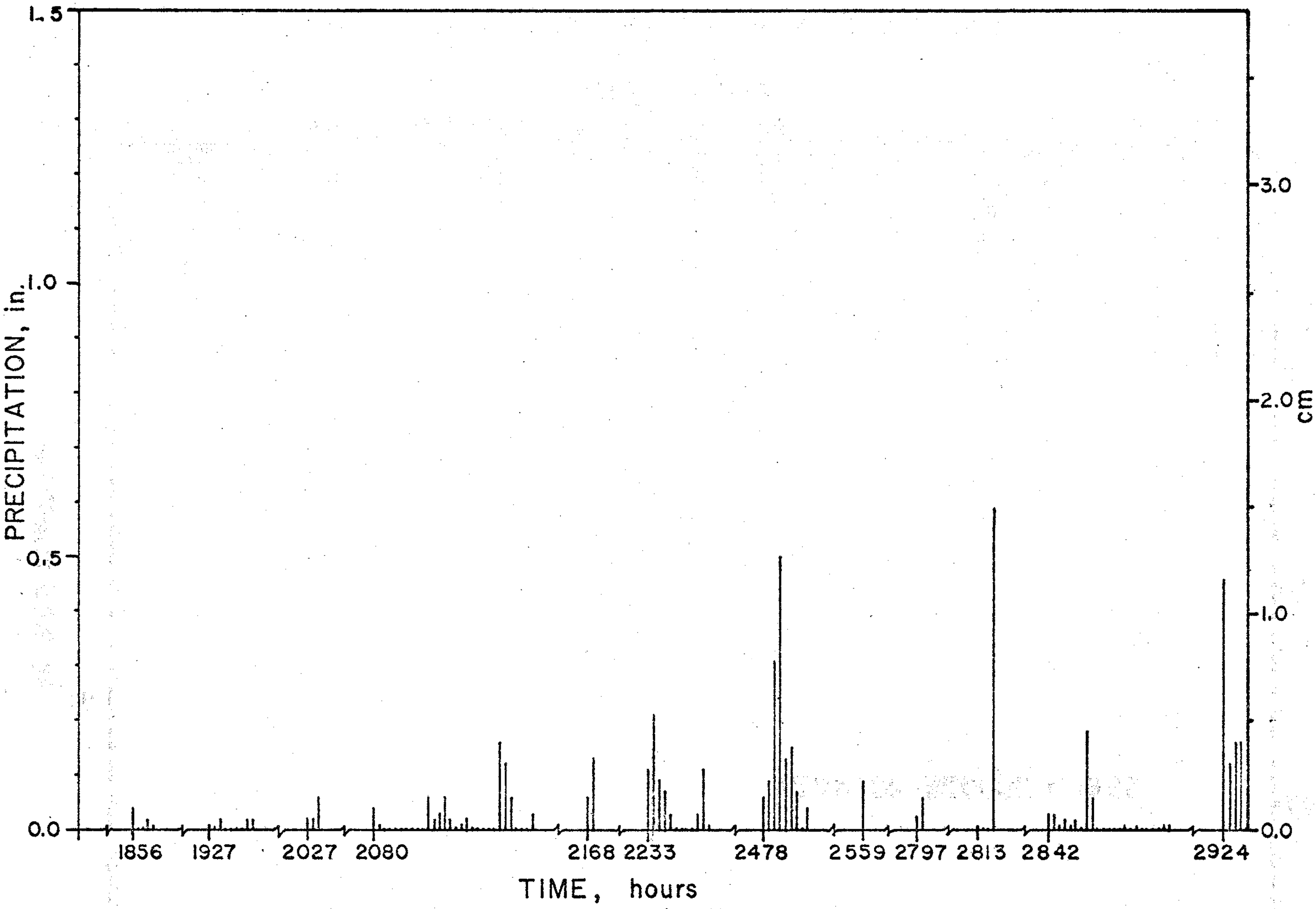

Figure VII-2 (continued). Point Rainfall for Des Moines, Iowa.

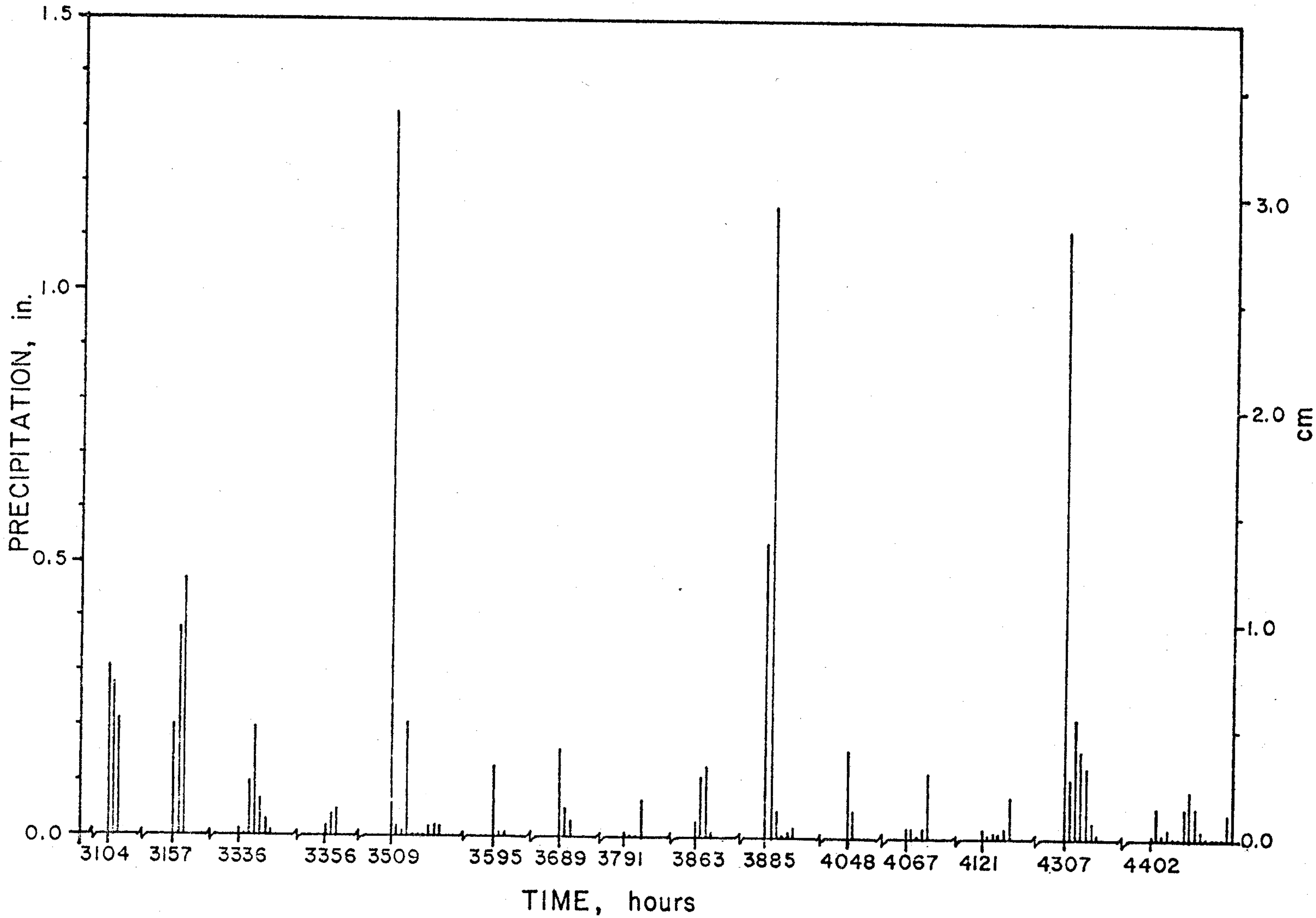

Figure VII-2 (continued). Point Rainfall for Des Moines, Iowa.

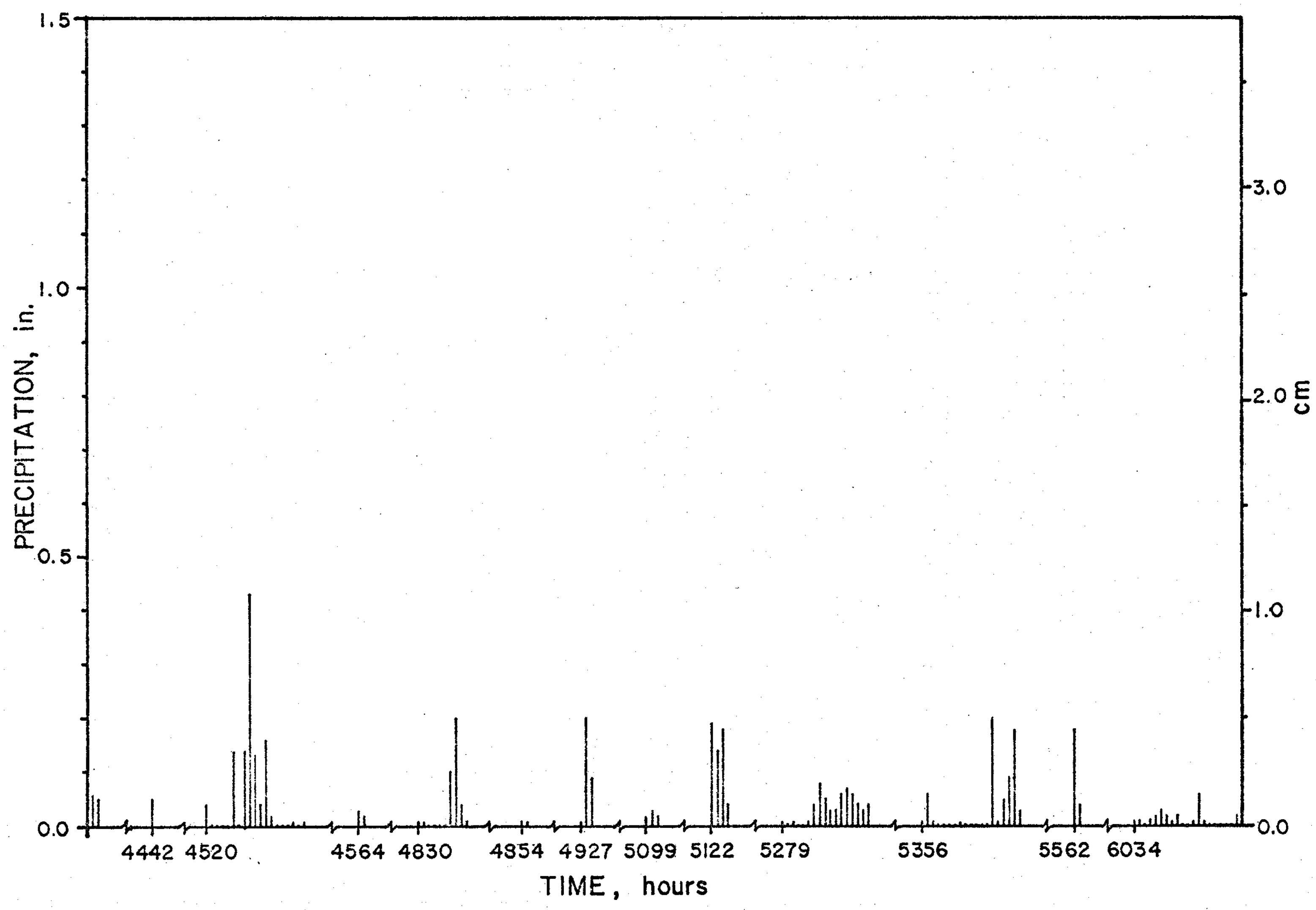

Figure VII-2 (continued). Point Rainfall for Des Moines, Iowa.

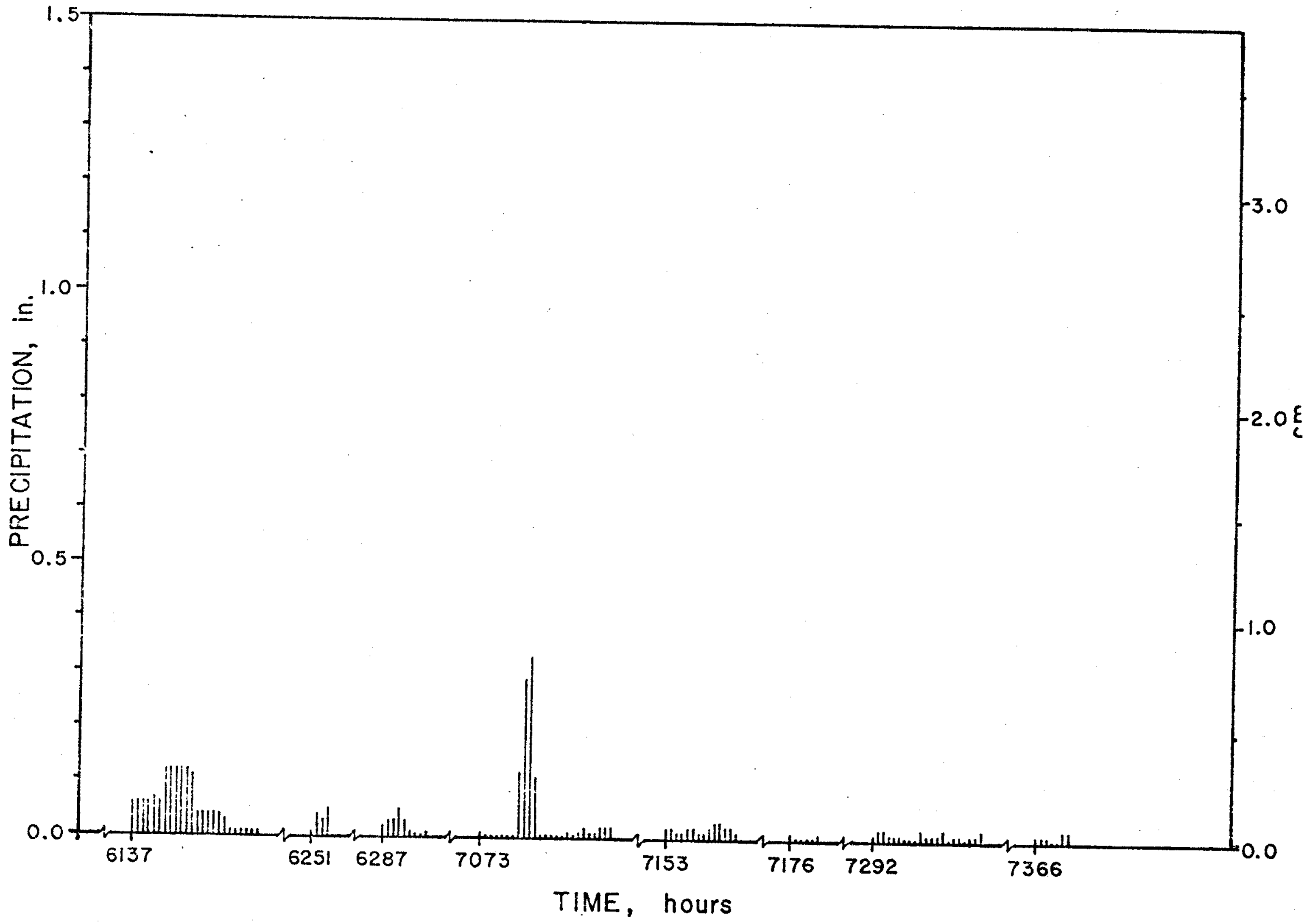

Figure VII-2 (continued). Point Rainfall for Des Moines, Iowa.

n = total number of data points or observations, and

k = number of hourly lags.

A plot of the serial correlation coefficients, r(k), against the number of lags, k, is called a correlogram. The technique of autocorrelation analysis is essentially a study of the behavior of the correlogram of the process under investigation.[12] The correlogram shape, or curve joining each point to the next, is henceforth referred to as the autocorrelation function. An analysis of the precipitation time series of Figure VII-2 results in the curve shown in Figure VII-3, Lag-k Autocorrelation Function of Des Moines, Iowa, Hourly Rainfall. At a lag of zero hours, the correlation of the discrete open series is unity because this point on the curve represents the linear dependence of the data series on itself. The number of observations (including zero values) totals 7,372 consecutive values, and lags up to 720 hours were investigated. The first minimum of the autocorrelation function occurs at a lag of 10 hours, and the value of the function is also zero at this point. The physical interpretation is that periods without rainfall for at least 10 hours separate uncorrelated, and therefore independent, storm events.

Actually any point of the autocorrelation function which lies outside of the 95 percent tolerance limits indicated in Figure VII-3 suggests a significantly non-zero correlation between storm events at that particular time lag. The Des Moines rainfall record obviously exhibits nonrandom behavior at lags of 377 hours ($\sim$ 16 days) and 421 hours ($\sim$ 18 days) in particular. The tolerance limits for a normal random time series of n values, and an open-series approach at a 95 percent probability level, are given by:[10]

$$\mathrm{TL}\ (95\%) = \frac{-1 \pm 1.645\ \sqrt{n-k-1}}{n-k} \qquad \text{(VII-6)}$$

where TL(95%) = tolerance limits at a 95% probability level.

As the number of lags, k, increases the tolerance limits diverge. However, the divergance is not noticeable for large n. Values of the autocorrelation function between lags of 100 to 300 hours and 500 to 720 hours fell between the 95 percent tolerance limits and are not shown in Figure VII-3.

Similarly, autocorrelation analysis was performed on the sequence of hourly runoff values generated by STORM from the rainfall input. The lag-k serial correlation coefficients, $r_0(k)$, are plotted against the number of lags in Figure VII-4, Autocorrelation Function of Hourly Urban Runoff for Des Moines, Iowa. The analytic technique establishes that the minimum interevent time of consecutive DWH that separates independent runoff events is 9 hours. Examination of Figure VII-4 reveals that the runoff time series is not purely random either. Linear dependence is observed at time lags of 377 hours ($\sim$ 16 days) and 436 hours ($\sim$ 18 days), as expected, because of the high correlation between rainfall and runoff processes.

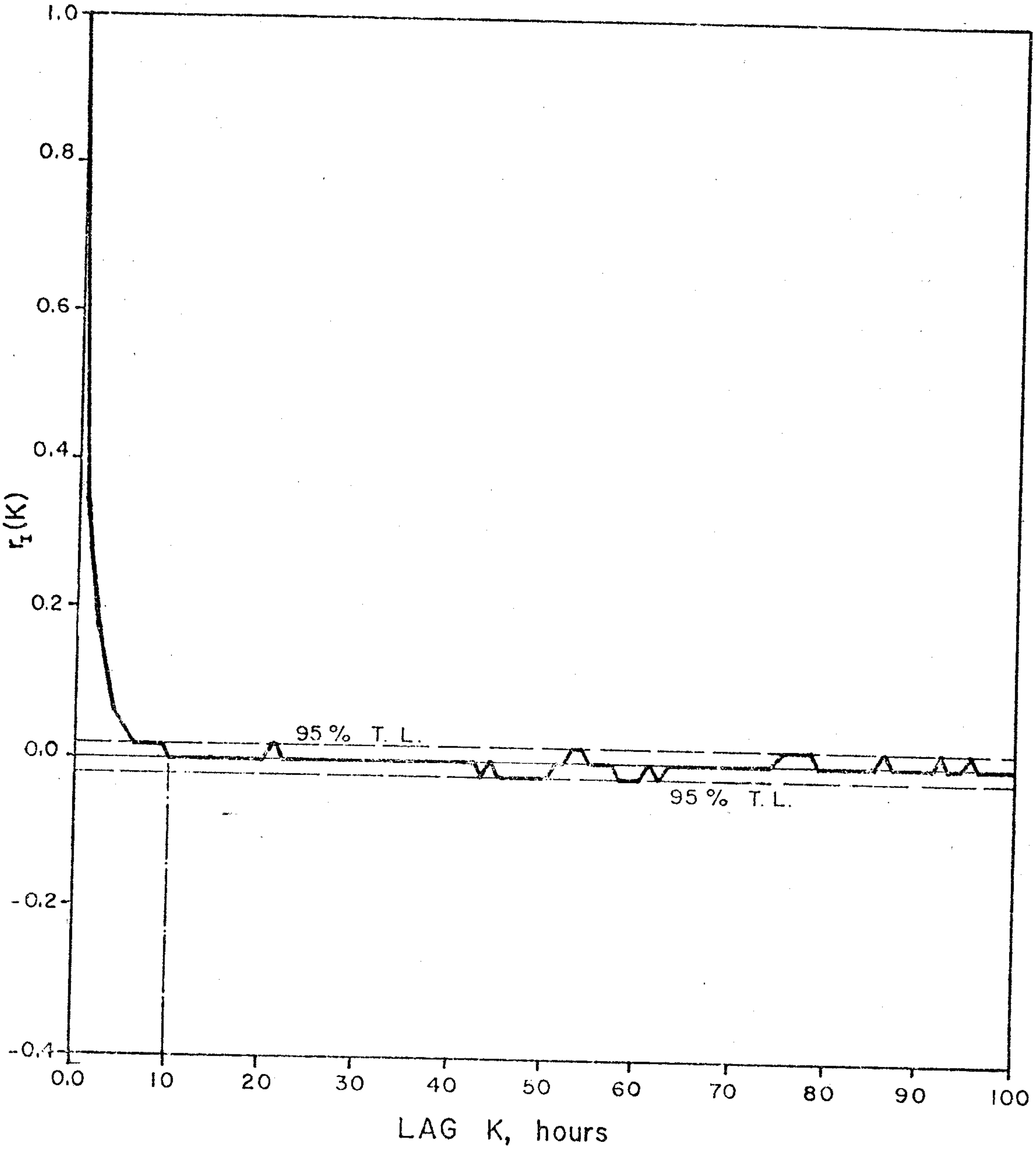

Figure VII-3. Lag-k Autocorrelation Function of Des Moines, Iowa, Hourly Rainfall, 1968.

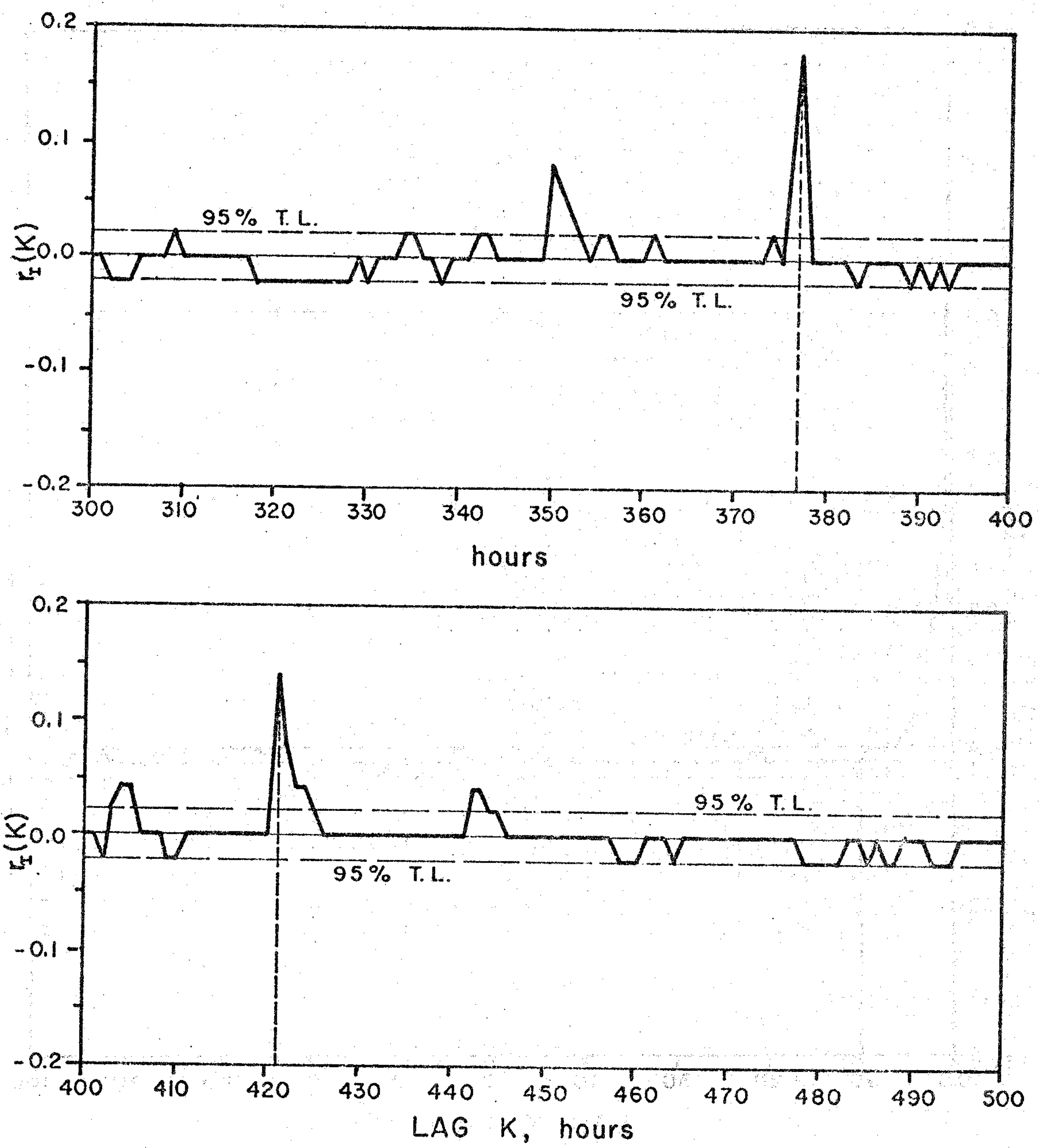

Figure VII-3 (continued). Lag-k Autocorrelation Function of Des Moines, Iowa, Hourly Rainfall, 1968.

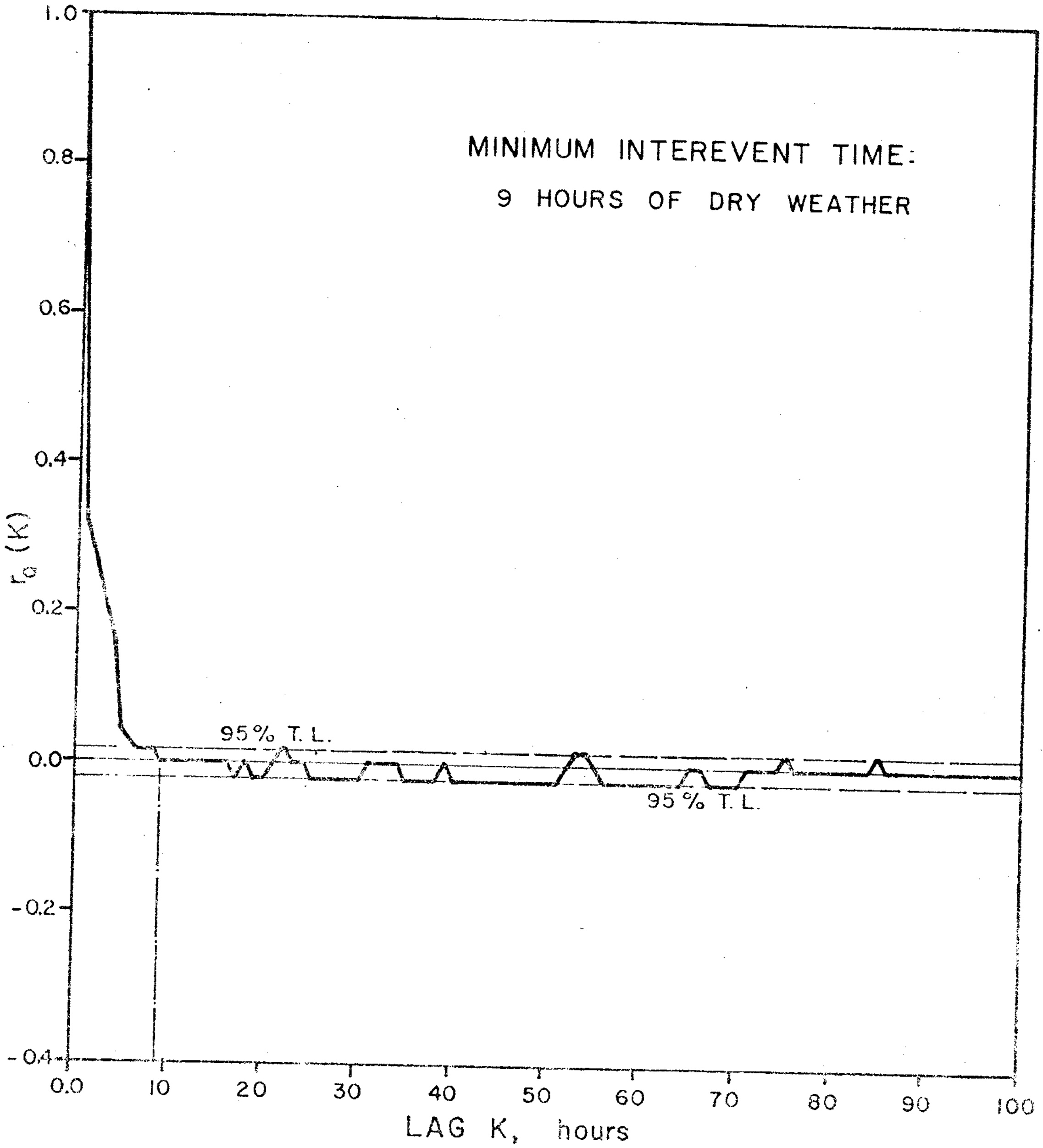

Figure VII-4. Autocorrelation Function of Hourly Urban Runoff for Des Moines, Iowa, 1968.

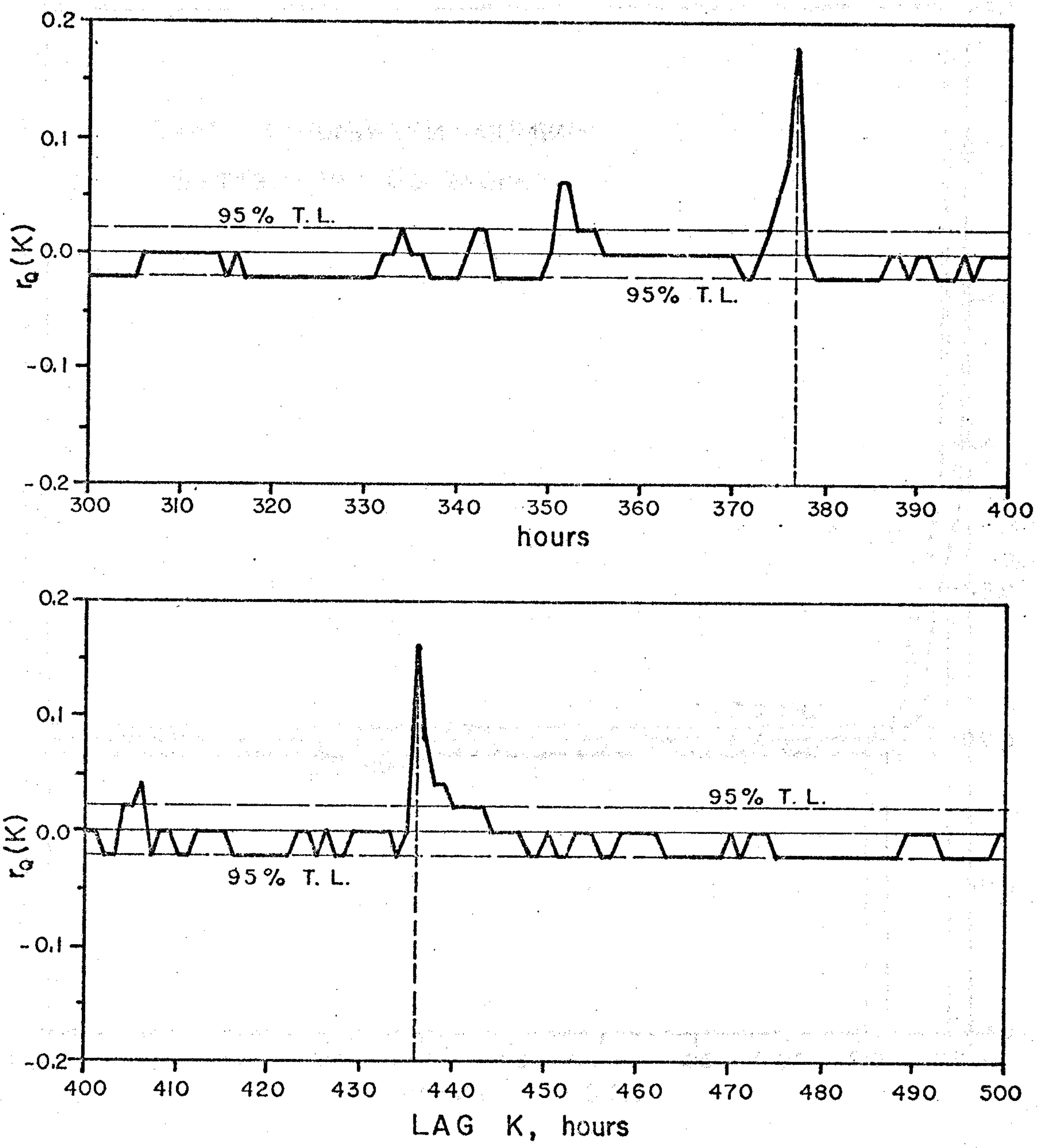

Figure VII-4 (continued). Autocorrelation Function of Hourly Urban Runoff for Des Moines, Iowa, 1968.

Slight differences are observed between the correlograms of Figures VII-3 and VII-4. These are due to the fact that depression storage and evaporation rates must be satisfied before runoff is generated by STORM. Thus, the digital simulation of the runoff process by STORM acts as a filter and has a slight smoothing effect.

The graphic procedure requires the number of dry-weather hours immediately preceding each hourly runoff occurrence. These values are determined directly by the chronological record provided by STORM of all the runoff events it generates from the input rainfall. If a hydrologic model such as STORM is not available, a close approximation may be obtained by assuming that the same numbers of DWH precede the rainfall and runoff events. Thus, the information provided by the precipitation records or the rainfall time series (such as Figure VII-2) is sufficient. A plot of the number of wet-weather events obtained by varying the minimum interevent time is shown in Figure VII-5, Definition of a Wet-Weather Event for Des Moines by Graphic Procedure. It is evident that a time value exists after which an increase in the minimum interevent time does not result in a correspondingly significant reduction in the number of storm events. The graphic procedure selects a period of 8 consecutive DWH as the minimum interevent time. The result is remarkably close to that obtained by the analytic technique, autocorrelation. In some cases, however, the graphical approach may not exhibit a curve with such a well-defined transition point. It is then necessary to apply classical statistical techniques to investigate the sequential properties of the hydrologic series.

Based on the above analyses, a wet-weather event and its duration are defined in the mathematical model as follows:

1. Any runoff occurrence having nine or more DWH preceding it denotes the beginning of the event (see below).

2. The event continues as long as all of the subsequent runoff occurrences have a DWH value immediately preceding them equal to or less than eight hours.

3. The event runoff duration (in hours) is equal to the sum of all the runoff occurrences in (2).

4. The actual event duration (in hours) must be determined by examining the date and hour of the first runoff value and the date and hour of occurrence of the last runoff value within the event.

The hourly urban runoff and associated pollutant loads within each event (including DWF pollutant loads during DWH periods less than nine hours duration) are summed, average conditions are determined, and the model proceeds with the receiving water analysis.

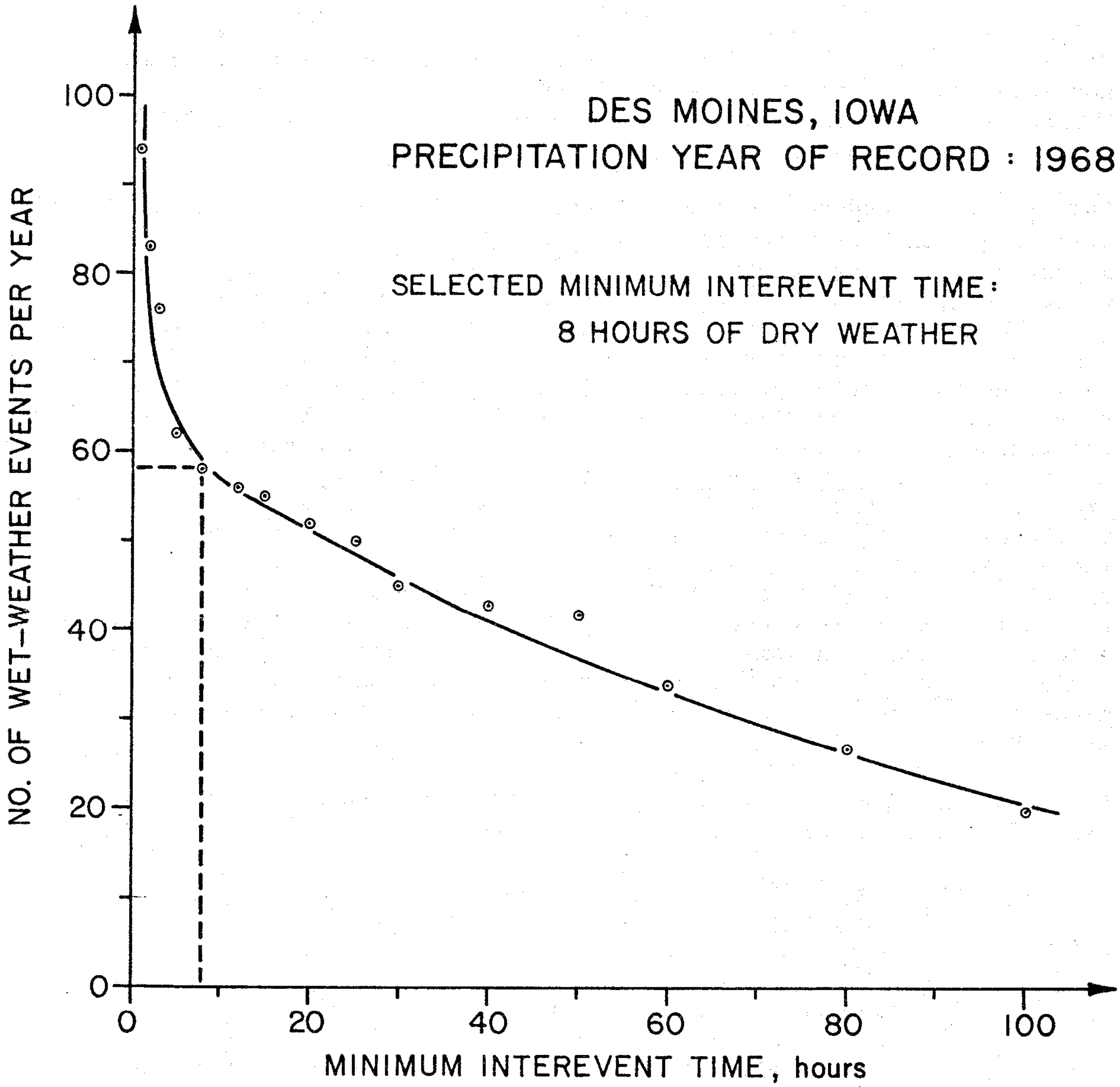

Figure VII-5. Definition of a Wet-Weather Event for Des Moines by Graphic Procedure

Separate Storm, Combined and Dry-Weather Loading

All of the following methodology can be used regardless of the technique employed to generate storm runoff and quality, as long as these values pertain to the entire area being modeled.

Separate Storm Flows and Loadings --

Apportionment of the total flow and BOD loading is made on the basis of the relative area served by separate and combined sewers. Runoff from separate sewered areas is thus (refer to Figure VII-1):

$$Q_s = \frac{A_s}{A_t} Q_t \qquad \text{(VII-7)}$$

where Q_s = stormwater flows from separate sewered areas, cfs,

A_s = area served by separate sewers, acres,

Q_t = total (storm plus combined) urban runoff, cfs, and

A_t = total area of catchment, acres.

The concentration of BOD in separate storm sewers, BOD_s, is simply the hourly value computed by STORM, BOD_t (mg/1), for the total urban runoff.

Dry-Weather Flow and Loadings --

Dry-weather flow and BOD loadings are assumed known from data on point sources in the area. Thus, Q_d represents the flow (cfs) into receiving waters of treated wastewater, and BOD_d represents the BOD concentration at 68°F (20°C) for 5 days, mg/1. The amount of treatment can be varied in the analysis, as stated earlier.

Combined Flows and Loadings --

Dry-weather flow (DWF) is assumed to cause only a negligible increase in flow in a combined sewer during a storm event. However, two factors related to DWF may increase significantly the BOD concentration of the combined sewer storm water:

1. the BOD strength of the municipal sewage with which it mixes; and

2. the BOD exerted by sediment accumulation in each section of the sewer under DWF conditions which is subject to the "first flush" effect induced by the initial runoff.

To incorporate the "first flush" effect, it is assumed that the hourly in-sewer sediment build-up is constant over consecutive dry-weather hours. This assumption is reasonable although it is evident that particle size and specific gravity, depth of flow, and the slope of the conduit are important factors affecting deposition. Data collected by Davis and Borchardt[9] at various combined sewer overflow stations in Des Moines, Iowa, support the first flush theory. BOD and total suspended solids (TSS) concentrations decreased with time with little or no relation to the flow pattern. Furthermore, pollutographs (BOD vs time and TSS vs time) for these stations seem to indicate that the flushing occurs mostly during the first hour of runoff generated by the storm event.

Thus, the mathematical model computes the sewer solids build-up that occurs during the consecutive DWH, then the BOD load contribution from these solids is lumped into the first hour of runoff. The first flush BOD load is given by

$$FF = FFLBS \cdot DWH \cdot A_c \qquad \text{(VII-8)}$$

where FF = first flush BOD load, lbs/hour,

FFLBS = first flush factor, lbs/first flush hour per DWH-acre,

DWH = number of dry-weather hours preceding each runoff event, and

A_c = area served by combined sewers, acres.

The first flush factor, FFLBS, must be determined from

1. the total flow generated by the combined sewer area (including dry-weather flow contribution) during the wet year;

2. the difference in annual average concentration between BOD_c (excluding factor FF) and the measured annual average value; and

3. the total number of DWH for the entire year under study.

An example of this calculation is presented in the application of the model to Des Moines, Iowa.

Apportionment of the total flow on the basis of relative area gives:

$$Q_c = \frac{A_c}{A_t} Q_t \qquad \text{(VII-9)}$$

where Q_c = combined sewer overflow rate, cfs.

Finally, the mixed BOD concentration in the combined sewer, BOD_c (mg/1), is computed by the following expression:

$$BOD_c = \frac{BOD_t \cdot Q_c + BOD_f \cdot Q_d \cdot (A_c/A_t) + FF \cdot C_1}{Q_c + Q_d \cdot (A_c/A_t)} \qquad \text{(VII-10)}$$

where C_1 = factor to convert FF from lb/hr to cfs · mg/1 = 4.45.

EFFECT ON STREAMS

Introduction

A broad spectrum of analyses is possible, ranging from simple examination of concentrations at the inflow points to use of detailed receiving water quality simulation models. A simplified mathematical modeling approach is used in which critical deficits and resulting minimum DO concentrations are determined for a large number of waste input combinations, stream conditions, and treatment schemes as indicated in the list of options given earlier. The construction of a detailed receiving water model is not justified for the problem context: to provide information on the relative adequacy of various pollutant control strategies for achieving selected water quality standards. In addition, a much larger data base would have been necessary on river channel geometry, continuous rather than discrete quantity and quality measurements, and other dynamic parameters. Many sophisticated models have outstripped the data base and have consequently severely limited their own applicability.

Some key assumptions, most of them typical of models for interim planning, are made:[13]

1. Temporal steady-state conditions prevail, where all system parameters and inputs are constant with respect to time; however, a relatively short time step (1 hour) is used for simulation.

2. Stream system parameters (such as river flow, velocity, depth, deoxygenation rate, and reaeration rate) are spatially constant along the flow axis throughout each time step.

3. All waste inflows occur at one point on the receiving stream.

4. The effects of various natural biological processes (algal photosynthesis and respiration, benthal stabilization) are incorporated into a background quality which is reflected by DO deficit (if none, by saturation) upstream from the waste inflow point. Any benthic buildup is incorporated into the BOD decay rate.

5. Waste treatment facilities operate at constant efficiencies, independent of hydraulic and organic loadings, for the entire period of simulation.

Initial Conditions

Initial conditions of BOD in the river are defined by equation VII-1. In subsequent equations, the mixed BOD concentration in the river will be denoted by L_o. Thus,

$$L_o \equiv BOD_m \tag{VII-11}$$

The assumption that all waste inflows occur at one point is not unreasonable for Des Moines, but in some locations the distribution of inflows along the river may need to be considered. It is important to emphasize that all of the BOD contributors in equation VII-1 represent BOD_5 values. Thus, the mixed BOD concentration in the river, BOD_m, is also in terms of the standard BOD test. The ultimate first-stage (carbonaceous) demand is related to the BOD_5 value by:

$$(L_o)_c = \frac{BOD_5}{1-e^{-5K_1}} \tag{VII-12}$$

where $(L_o)_c$ = ultimate first-stage BOD demand, mg/l, and

K_1 = first-order BOD decay rate constant, day^{-1}.

Through verification analysis the value of K_1 was determined to . . 0.7 day^{-1}. Thus, for Des Moines,

$$(L_o)_c = 1.03\ BOD_5 \tag{VII-13}$$

and the conversion is unnecessary. Of course, generalizations cannot be made because the decay rate may vary considerably for different river systems.

The other initial condition required is the initial oxygen deficit, D_o. It is assumed that all waste inflows will be at saturation. Thus, the only contribution to the initial deficit will be from the upstream portion of the river. Thus,

$$D_o = \frac{D_u Q_u}{Q_u + Q_d + Q_s + Q_c} \qquad \text{(VII-14)}$$

where D_o = initial DO deficit, mg/1, and

D_u = DO deficit in receiving waters upstream of inflow point, mg/1.

Oxygen Balance of Polluted Streams

Pollutant transport processes in a stream system may be adequately approximated by the one-dimensional version of the classical convective diffusion equation. This partial differential equation is based on the principles of conservation of mass (continuity) and is given by:

$$\frac{\partial C}{\partial t} = \frac{\partial}{\partial x}\left[E\frac{\partial C}{\partial x} - UC\right] \pm \Sigma S \qquad \text{(VII-15)}$$

where C = concentration of water quality parameter (pollutant), M/L^3,

t = time, T,

$-E\frac{\partial C}{\partial x}$ = mass flux due to longitudinal dispersion along the flow axis, the x direction, M/L^2T,

UC = mass flux due to advection by the fluid containing the mass of pollutant, M/L^2T,

S = sources or sinks of the substance C, M/L^3T,

U = flow velocity, L/T, and

E = longitudinal dispersion coefficient, L^2/T.

The equation assumes no diffusion of pollutants through the river boundaries (other than what is included in the source-sink term) and is best suited to predict concentrations relatively far downstream from the point of waste injection. Since critical DO deficits usually occur some distance downstream from the waste source, equation VII-15 is particularly well suited for such predictions.

The main sources of dissolved oxygen in the stream are atmospheric reaeration and oxygen production by photosynthesis. The major sinks include carbonaceous oxygen demand (CBOD), nitrogenous oxygen demand

(NBOD), benthal demand, and respiration of aquatic plants. All stream system parameters are assumed spatially constant along the flow axis, and by substituting the various sources and sinks of DO into equation VII-15 the following expression is obtained:

$$\frac{\partial C}{\partial t} = E\frac{\partial^2 C}{\partial x^2} - U\frac{\partial C}{\partial x} + K_2(C_s - C) \qquad \text{(VII-16)}$$

$$- K_1L - K_nN_n + P - R_e - B$$

where

C = concentration of DO in the stream, mg/1

E = longitudinal dispersion coefficient, ft^2/sec,

U = freshwater stream velocity, ft/sec,

K_2 = atmospheric reaeration coefficient, $hours^{-1}$,

C_s = dissolved oxygen saturation, mg/1,

C_s-C = dissolved oxygen deficit, mg/1 = D,

K_1 = deoxygenation constant of carbonaceous BOD, $hours^{-1}$,

L = remaining carbonaceous BOD concentration, mg/1,

K_n = oxidation coefficient of nitrogenous BOD, $hours^{-1}$,

N_n = remaining nitrogenous BOD concentration, mg/1,

P = oxygen production rate by algal photosynthesis, mg/1-hour,

R_e = algal respiration rate, mg/1-hour, and

B = benthal demand of bottom deposits, mg/1-hour.

For freshwater streams, the advective flux is significantly larger than the mass flux due to longitudinal dispersion.[13] For steady-state analysis, all system parameters are assumed constant in time. Since it is desired to solve for the DO deficit and

$$\frac{\partial C}{\partial t} = 0;\; E = 0;\; \frac{\partial C}{\partial x} = -\frac{\partial D}{\partial x} \qquad \text{(VII-17)}$$

equation VII-16 reduces to an ordinary differential equation:

$$0 = U\frac{dD}{dx} + K_2D - K_1L - K_nN_n + P - R_e - B. \qquad \text{(VII-18)}$$

A basic assumption of the model, stated earlier, is that the effects of the biological processes ($P - R_e - B$) are incorporated into the measured upstream DO deficit. The production and consumption of oxygen by these processes after waste injection are assumed negligible since no data were available. Although measurements of total organic nitrogen and nitrate-nitrogen for upstream sources are available, modeling of the nitrogenous BOD was not possible because:

1. the amount of organic and ammonia nitrogen present in all wastewater inputs is unknown; and

2. the oxidation coefficient of nitrogenous BOD is also unknown.

By applying the above assumptions and expressing the remaining carbonaceous BOD, L, in terms of the ultimate carbonaceous BOD and its reaction, the governing differential equation for dissolved oxygen deficit is reduced to the expression:

$$0 = U\frac{dD}{dx} + K_2D - K_1L_oe^{-\frac{K_1x}{U}} . \qquad \text{(VII-19)}$$

Critical Deficit and DO Levels

The solution of equation VII-19 constitutes the Streeter-Phelps formulation in which the deficit as a function of time since release is

$$D = \frac{K_1L_o}{K_2 - K_1}\left(e^{-K_1t} - e^{-K_2t}\right) + D_oe^{-K_2t} \qquad \text{(VII-20)}$$

where D = DO deficit, mg/l,

K_1 = deoxygenation coefficient, hours^{-1},

K_2 = reaeration coefficient, hours^{-1}, and

t = elapsed time, hours.

The critical (maximum) deficit is found through differentiation to be

$$D_c = \frac{K_1L_o}{K_2}e^{-K_1t_c} \qquad \text{(VII-21)}$$

where D_c = critical (maximum) oxygen deficit, mg/1, and

t_c = elapsed time at which critical deficit occurs, hours.

The value of t_c is given by

$$t_c = \frac{1}{K_1(f-1)} \ln \left\{ f[1 - (f-1)\frac{D_o}{L_o}] \right\} \qquad \text{(VII-22)}$$

where f = self-purification ratio

$= K_2/K_1$.

The minimum DO level is calculated as

$$C_{min} = C_s - D_c \qquad \text{(VII-23)}$$

where C_{min} = concentration of DO at maximum deficit, mg/1, and

C_s = saturation concentration of DO, mg/1.

The saturation concentration is determined from the regression relationship developed by ASCE,[14]

$$C_s = 14.652 - 0.41022T + 0.0079910T^2 - 0.000077774T^3 \qquad \text{(VII-24)}$$

where T = water temperature, °C.

Values of K_1 and K_2

The deoxygenation coefficient, K_1, represents the loss of DO in the water due to reduction of BOD. A calibrated value of 0.7 day^{-1} is used for this simulation for K_1 at 20°C (68°F). A temperature correction and conversion to $hour^{-1}$ gives:

$$K_1(T) = \frac{1}{24}K_1(20°)1.047^{T-20} \qquad \text{(VII-25)}$$

A variety of formulas exists for prediction of the reaeration coefficient K_2, almost all of which depend upon velocity, U, and depth, H. The equation of Langbein and Durum[15] was chosen because it is most closely related to subsequent procedures used to obtain U and H.

$$K_2 = 2.303 \left[3.3 \frac{U}{H^{1.33}} \right] \qquad \text{(VII-26)}$$

where K_2 = reaeration coefficient at 20°C, day^{-1},

U = stream velocity, ft/sec, and

H = stream depth, ft.

The problem lies in obtaining values of U and H, since the streamflow varies with time. In the absence of measurements, or if the data cannot be obtained in an expedient manner (as in the ensuing application to the Des Moines River), an approximation can be made based on the work of Leopold and Maddock[16] in which they show strong correlations between velocity vs flow and depth vs flow, namely:

$$U = \alpha_1 Q^{\alpha_2} \qquad \text{(VII-27)}$$

$$H = \beta_1 Q^{\beta_2} \qquad \text{(VII-28)}$$

where Q = streamflow, cfs, and

$\alpha_1, \alpha_2, \beta_1, \beta_2$ = regression coefficients.

The model presently utilizes coefficients which were determined for the Kansas River System in Kansas and Nebraska, for which

$$\alpha_1 = 1.60 \qquad \text{(VII-29a)}$$

$$\alpha_2 = 0.03 \qquad \text{(VII-29b)}$$

$$\beta_1 = 0.11 \qquad \text{(VII-29c)}$$

$$\beta_2 = 0.45. \qquad \text{(VII-29d)}$$

When equations VII-27, VII-28 and VII-29 are substituted into VII-26 and conversion is made to units of hour^{-1}, the reaeration coefficient is established as a function of streamflow, Q:

$$K_2 = 2.303 \cdot 4.1435\ Q^{-0.57} \cdot 1.024^{T-20} \qquad \text{(VII-30)}$$

where T = stream temperature, °C, and the last factor represents a temperature correction.

Total Volume of DO Deficit

One reason for determining DO levels for several inflow options is to attempt to establish the relative effect of one source versus another. Unfortunately, the critical deficit, equation VII-21, is nonlinear in terms of the initial BOD, L_o, thus effects cannot be separated directly. One measure of the relative importance is the integral, or summation of the deficit equation VII-20 over all time,

$$\forall = \int_o^\infty D\ dt = \frac{D_o + L_o}{K_2} \qquad \text{(VII-31)}$$

where $\forall$ may be interpreted as the total volume of deficit, with units of mg-hours/l. Values of $\forall$ are displayed by the model for each inflow combination.

Presentation of Results

The impact of urban runoff on the receiving waters is evaluated in terms of violations of potential DO standards, i.e., by the number of times predicted DO levels fall below a specified value. The receiving program, running on an hourly time step for rainy events only, maintains a tabulation of the frequency of DO values within specified intervals, from which the cumulative relative frequency may be plotted, as sketched in Figure VII-6, Hypothetical Results of Simulation. For example, from the figure, the percent of wet-weather events during which a DO standard of, say, 4 mg/l is violated may be readily obtained. Different input options will produce different curves in Figure VII-6 and they may be compared in this manner. Curves are also developed for dry-weather periods and a cumulative annual frequency is obtained.

In addition, the total "volume" of DO deficit, equation VII-31, may be compared for each option. This will give some indication of the relative impact of one option versus another, although the results have no special physical significance.

APPLICATION TO DES MOINES, IOWA

General Description

The City of Des Moines, Iowa is located near the confluence of the Des Moines River and the Raccoon River as shown in Figure VII-7, Map of Des Moines Area. It contains approximately 200,000 people out of the total of 288,000 for the metropolitan area.[9] The mean annual precipitation is 31.27 inches (795 mm) which is approximately equal to the United States average. Based on an extensive sampling program, annual pollutant unit loads upstream from the city were determined (see Table VII-3, Pollutant Unit Loads

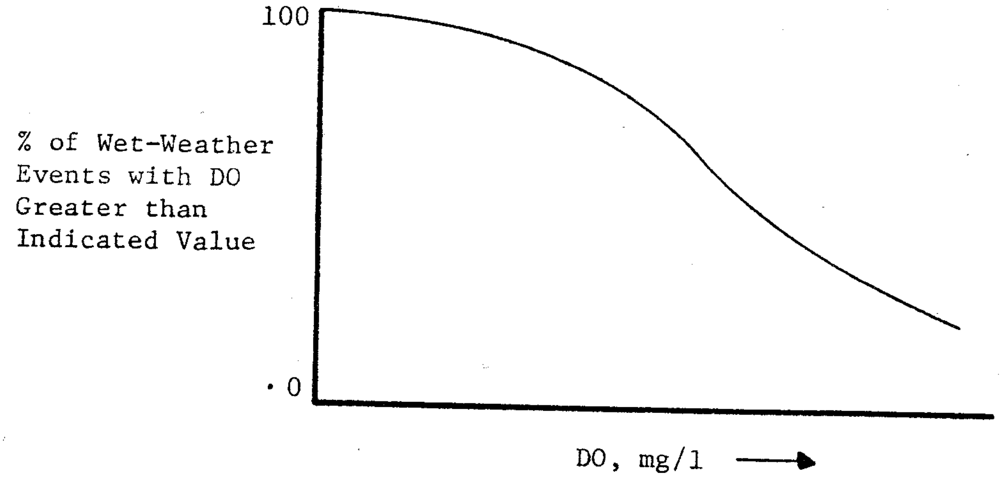

Figure VII-6. Hypothetical Results of Simulation

Table VII-3. POLLUTANT UNIT LOADS FOR DRAINAGE AREA[a] ABOVE DES MOINES, IOWA (Davis and Borchardt, 1974)[9]

	Des Moines River	Raccoon River	Total
Drainage Area, acres (ha)	3,738,000 (1,512,769)	2,202,000 (891,149)	5,940,000 (2,403,918)
Unit Average Annual Runoff, acre-ft/acre (ha-m/ha)	0.42 (0.13)	0.40 (0.12)	0.41 (0.12)
Unit BOD, lbs/acre (kg/ha)	13.40 (15.02)	6.93 (7.77)	11.01 (12.34)
Unit NO_3, lbs/acre (kg/ha)	3.75 (4.20)	3.74 (4.19)	3.75 (4.20)
Unit PO_4, lbs/acre (kg/ha)	0.54 (0.61)	0.42 (0.47)	0.50 (0.56)

[a]On an annual basis.

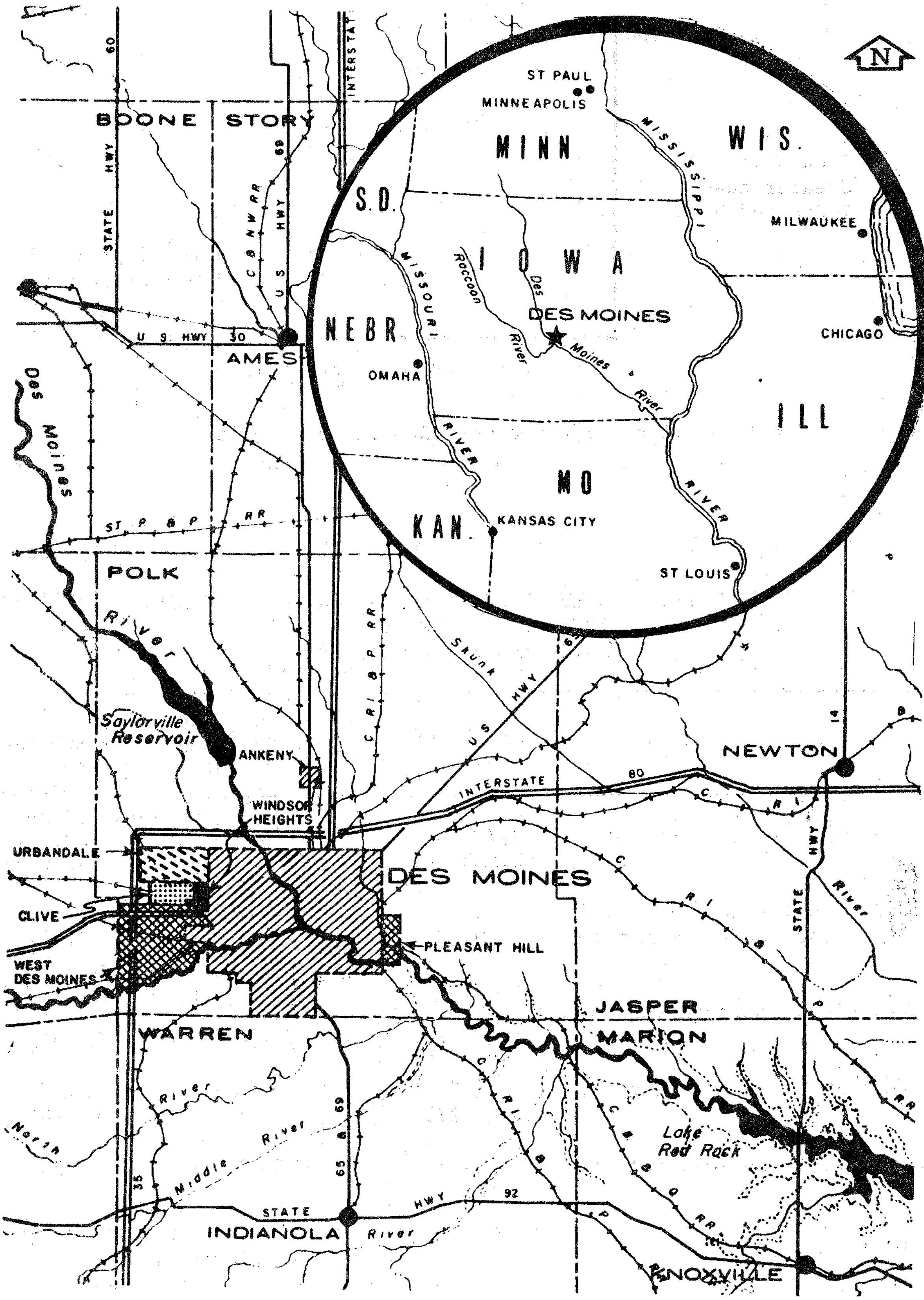

Figure VII-7. Map of Des Moines Area (Davis and Borchardt, 1974)[9] for Drainage Area Above Des Moines, Iowa). The estimated annual loading from the urban area's 45,000 acres of separate sewer and 4,000 acres of combined sewer systems is shown in Table VII-4, Summary of Present Annual Metro Area Discharges.

Taking the total upstream drainage area for the Raccoon and Des Moines Rivers, the pollutant contributions are: 65,225,000 pounds of BOD (29,586,000 kg); 22,222,000 pounds of NO_3 (10,080,000 kg); and 2,940,000 pounds of PO_4 (1,334,000 kg). The urban area loadings (when added to upstream values) represent, respectively: 15 percent, 3 percent and 51 percent of the total BOD, NO_3, and PO_4 mass loadings to the Des Moines River below the metropolitan area. The Davis and Borchardt report estimates made from river sampling data taken below Des Moines indicate the following average annual river loadings: 70,000,000 pounds of BOD (31,751,466 kg); 25,400,000 pounds of NO_3 (11,521,250 kg); and 7,950,000 pounds of PO_4 (3,606,059 kg). These figures reveal that: (1) 6,610,000 pounds of BOD (2,998,246 kg) are "lost" in transit through the urban section of the stream, and (2) by contrast 2,474,360 pounds of NO_3 (1,122,351 kg) and 1,917,400 pounds of PO_4 (869,718 kg) are gained in addition to the measured urban sources.

Davis and Borchardt offer some explanations:

> The "sometimes" decrease in organic load through the metro area may be attributable to treatment realized in the low head impoundments at Scott and Center Streets on the Des Moines River and just below Fleur Drive on the Raccoon. To some extent these impoundments may be serving as intermittent sedimentation and stabilization units.
>
> All BOD data, including that used from the two other studies, were obtained from unfiltered samples. However, since the analytical technique was the same for all samples, the relative magnitude of the data should not be affected.
>
> There has been some speculation that treated wastewater effluents may exert an antagonistic or retardant effect on the BOD exertion rate of the receiving stream. If true, this may be due to surfactants or to the expected lower exertion rate of the effluent. In this regard, the decreased BOD in 4 or 5 measurements between R-5 and R-6 is of interest. Increased loads between the summation of R-4 and R-9 versus R-5 are likely due to raw and combined sewage bypassing the intervening area.
>
> Another, and probably the most practical, possibility for the discrepancies is the fact that the data are biological and biochemical in nature and such data do not always provide predictable comparative summations.

Table VII-4. SUMMARY OF PRESENT ANNUAL METRO AREA DISCHARGES[a] (Davis and Borchardt, 1974)[9]

	Days	BOD, lbs (kg)	NO_3, lbs (kg)	O.PO_4, lbs (kg)
Wastewater Treatment Plant Effluent				
Dry-Weather	257	4,060,600 (1,841,857)	400,900 (181,845)	1,737,300 (788,026)
"Wet" Dry-Weather[b]	108[e]	2,246,400 (1,018,950)	237,600 (107,774)	1,036,800 (470,285)
Subtotal	365	6,307,000 (2,860,807)	638,500 (289,619)	2,774,100 (1,258,311)
"Wet" Dry-Weather Overflow[c]	108[e]	2,235,600 (1,014,051)	9,700 (4,400)	263,500 (119,522)
Wet-Weather Combined Sewer Overflows[d]				
2.72 in. (69.1 mm) Rain	1	40,500 (18,370)	240 (109)	6,350 (2,880)
1.50 in. (38.1 mm) Rain	5	101,500 (46,040)	680 (308)	12,200 (5,534)
0.75 in. (19.1 mm) Rain	12	32,500 (14,742)	220 (100)	3,250 (1,474)
0.375 in. (9.5 mm) Rain	18	0	0	0
0.175 in. (4.4 mm) Rain	20	0	0	0
Subtotal	56	174,600 (79,197)	1,140 (517)	21,800 (9,888)

Table VII-4 (continued). SUMMARY OF PRESENT ANNUAL METRO AREA DISCHARGES

	Days	BOD, lbs (kg)	NO_3, lbs (kg)	$O.PO_4$, lbs (kg)
Urban Storm Water Discharges[d]				
2.72 in. (69.1 mm) Rain	1	292,000 (132,449)	6,800 (3,084)	3,900 (1,769)
1.50 in. (38.1 mm) Rain	5	765,000 (346,998)	15,300 (6,940)	9,200 (4,173)
0.75 in. (19.1 mm) Rain	12	966,000 (438,170)	19,300 (8,754)	12,000 (5,443)
0.375 in. (9.5 mm) Rain	18	495,200 (224,619)	9,900 (4,491)	6,200 (2,812)
0.175 in. (4.4 mm) Rain	20	149,800 (67,948)	3,000 (1,361)	1,900 (862)
Subtotal	56	2,688,000 (1,219,256)	54,300 (24,630)	33,200 (15,059)
Total Annual Discharge	365	11,385,100 (5,164,194)	703,640 (319,166)	3,092,600 (1,402,780)

[a]Based on sampling periods from October 1968 to October 1969.

[b]Based on hydraulic capacity of treatment plant and its average effluent BOD during wet-weather periods from May to July 1969.

[c]Refers to sanitary overflow during wet-weather.

[d]Based on annual average rainfall patterns obtained from NOAA.

[e]Includes severe infiltration which prolonged effects of wet weather.

The sampling stations (Numbers 4, 5, 6, 9) are shown in Figure VII-8, Location Map: River Sampling Points. Intervening creeks, such as Beaver Creek, which carries nutrient loads of 2,860,000 pounds of NO_3 per year (1,297,274 kg per year) and 390,000 pounds of PO_4 per year (176,900 kg per year), may be an answer to observed differences in the nitrate loads. However, the phosphate totals are still unbalanced and the cause unresolved.

As stated earlier, the relative importance of a city is expected to increase as the size of the upstream drainage area decreases. To simulate this effect the model application to Des Moines investigates the response of the receiving water when upstream river flow, Q_u, is reduced to various fractions of measured flow. This option and others used for the Des Moines simulations are summarized in Table VII-5, Options Used for Des Moines Simulations. Thus, there is output for $5 \cdot 4 \cdot 3 \cdot 3 = 180$ combinations for each of four different combined area fractions. Conditions particular to Des Moines, Iowa, and the Des Moines River are investigated as well as the response of the receiving stream to hypothetical situations.

Data Sources

The data may be broken into categories describing needs for the runoff simulation. All land use, population density, areas, curb lengths, etc., were obtained from data prepared by APWA for STORM simulations (Volume III). Hourly rainfall values for the year 1968 were obtained from the National Weather Records Center at Asheville, North Carolina. The area served by combined sewers, A_c = 4,000 acres, is given on p. 2 of the Davis and Borchardt report.[9] Dry-weather flow values are taken from Table 5, p. 52. Receiving water upstream flows, temperatures, BOD and DO levels are taken from pp. 285-308. Total urban runoff (Q_t) and its BOD concentration (BOD_t) are obtained from the STORM simulation on an hourly basis. BOD_s, Q_s, BOD_d, Q_d, BOD_c, BOD_w and Q_w are computed from input data and appropriate mixing equations by the simplified mathematical model described in this section.

The first flush factor, FFLBS, was determined as follows:

$$\text{DWH/year} = 6{,}993 \text{ hour}$$

$$\text{Total flow combined} = 1.55 \times 10^{8} \text{ cf/yr} \quad (4.39 \times 10^{6} \text{ cu m/yr}).$$

Mixed concentration of storm water (from STORM) plus DWF = BOD_c = 62 mg/1.

The annual average BOD concentration in the combined sewer was measured by Davis and Borchardt[9] to be 72 mg/1.

$$\text{BOD difference} = 10 \text{ mg/1} = 0.0006243 \text{ lbs/ft}^3$$

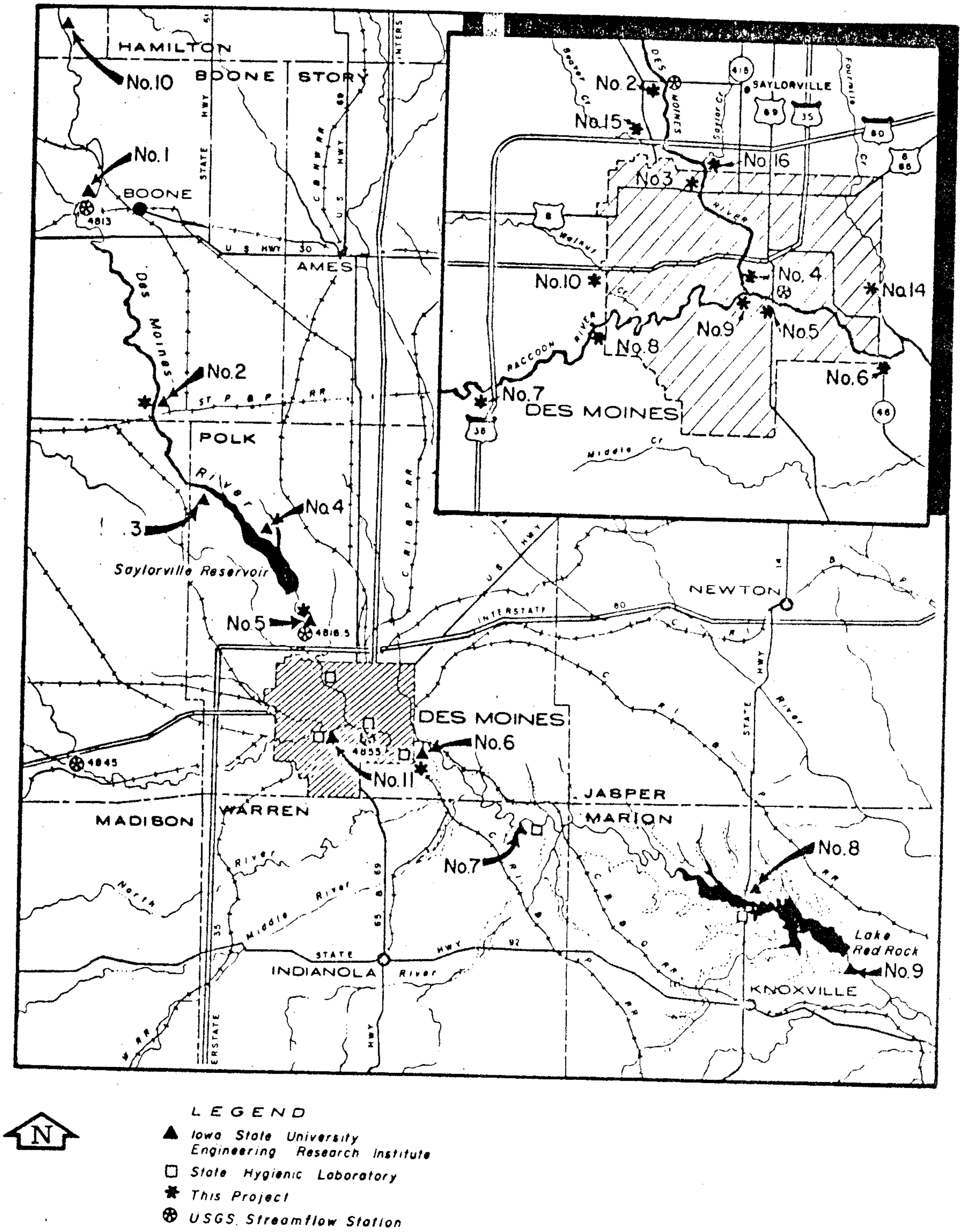

Figure VII-8. Location Map: River Sampling Points (Davis and Borchardt, 1974)[9]

TABLE VII-5. OPTIONS USED FOR DES MOINES SIMULATIONS

The following five inflow combinations (M)[a] are used:

1. River flow + DWF
2. River flow + DWF + separate flow
3. River flow + DWF + combined flow
4. River flow + separate flow + combined flow
5. River flow + DWF + separate flow + combined flow

and the following four DWF treatment rates (J) are used:

1. 0% (no treatment)
2. 30% (primary)
3. 85% (secondary)
4. 95% (tertiary)

and the following three WWF treatment rates (L) are used:

1. 0% (no treatment)
2. 25%
3. 75%

and the following three river flows (K) are used:

1. 0% of measured flow
2. 25% of measured flow
3. 100% of measured flow

and the fraction of combined area is varied four times:

1. 0% of total urban area
2. 8.16% of total urban area (existing conditions)
3. 50% of total urban area
4. 100% of total urban area

---

[a]Parameters J, K, L and M were used as subscripts in the computer output and serve to aid in labeling the various combinations.

$$\text{BOD load} = (0.0006243 \text{ lbs/cf})(1.55\text{x}10^8 \text{ cf/yr})$$

$$= 96{,}766.50 \text{ lbs/yr } (43{,}892.50 \text{ kg/yr})$$ that can be attributed to first flush effects

$$\text{FFLBS} = \frac{96{,}766.50 \text{ lbs/yr}}{(6993 \text{ DWH/yr})(4000 \text{ acres})}$$

$$= 0.0035 \text{ lbs/DWH-acre}$$

$$= 0.0039 \text{ kg/DWH-ha.}$$

This factor, as demonstrated earlier, is then used in equation VII-8 to estimate the first flush BOD load, FF, during the first hour of runoff generated by each storm event.

Special Problems

During the model application to Des Moines, various problems were encountered revolving around the critical deficit (D_c) and critical time (t_c) equations, equations VII-21 and VII-22, respectively. Due to the large number of conditions being simulated, including dry watercourses in which the waste inputs constituted the only flow, situations were encountered in which:

1. the deficit load ratio, R_o, was undefined

$$R_o = \frac{D_o}{L_o} \qquad \text{(VII-32)}$$

   because both D_o and L_o were equal to zero;

2. the self-purification ratio, f, was equal to one, causing equation VII-22 to be undefined; and

3. values of R_o were such that negative values of t_c were obtained.

Mathematical analysis led to the incorporation of certain modifications and safeguards. Thus, equations VII-21 and VII-22 were defined only for:

1. $f \neq 1$,

2. $L_o \neq 0$, and

3. $0 \leq R_o \leq 1/f$.

Otherwise, in order, if

1. $f = 1$, then

$$D_c = L_o e^{R_o - 1} \qquad \text{(VII-33)}$$

2. $f \neq 1$, $L_o = 0$, then

$$D_c = D_o \qquad \text{(VII-34)}$$

3. $f \neq 1$, $L_o = 0$, $R_o > 1/f$, then

$$D_c = D_o. \qquad \text{(VII-35)}$$

These equations, obtained by taking limits, are not particular to Des Moines and are applicable to any receiving stream.[17]

Verification Analysis

An important part of the total effort required to develop a mathematical model of water quality in a stream is devoted to verification and improvement of model accuracy. The verification procedure recommended for steady-state water quality models includes:

1. examination of model output using preliminary coefficients on a diverse set of data (different waste loads and temperatures under conditions of high and low flow, and variable initial stream quality);

2. assessment of the closeness of fit of observed field data to computed values;

3. adjustment of the model coefficients until the desired accuracy is obtained; and

4. achievement of a mathematical abstraction that reasonably reproduces observed stream response and establishes the necessary validity for planning purposes.

The verification procedure was preceded by calibration of the urban runoff BOD_5 loading rates for Des Moines, Iowa, as computed by STORM. The dust and dirt surface loading factors were adjusted to obtain an annual average BOD_5 concentration of 53 mg/l for urban stormwater runoff. The above concentration was the average value determined by the field monitoring program in the separate sewer system.[9] The developed mathematical model, as discussed in the methodology, simulates the mixing of stormwater runoff and sanitary sewage in the combined sewer system. The annual average BOD_5 concentration of combined sewer overflows was computed to be 75 mg/l, including the effects of first flush. The average value determined by the field monitoring program in the combined sewer system was determined to be 72 mg/l.[9]

The carbonaceous BOD reaction coefficient, K_1, was refined during the verification process to a final value of 0.70 day^{-1} (at 20°C). The model, of course, converts to units of $hour^{-1}$ and adjusts for temperature through equation VII-25. The atmospheric reaeration coefficient, K_2, is calculated internally as a function of streamflow and temperature by equation VII-30; therefore, no adjustment was necessary. Measured and computed values of DO at a point 5.6 miles (9.0 km) downstream from the confluence of the Raccoon and Des Moines Rivers are compared in Figure VII-9, Application to Des Moines, Iowa. Correlation between the calculated and observed profiles is quite good. The point corresponds to sampling location no. 6 as shown previously in Figure VII-8.

Included in Figure VII-9 are rainfall and average total river flow values for each wet-weather event (as defined in Figure VII-4). Differences between measured and computed DO concentrations may be attributed to such factors as: (1) the time of day during which the sample was taken; (2) the lag time between sampling and laboratory analysis and the temperature variations in the receiving water during the day; and (3) a lack of data on photosynthesis, algal respiration, and benthic demand. The time scale in days represents the wet year beginning on March 8 and ending December 30, 1968. Again, it should be reemphasized that these DO values are not the minimum DO's resulting from maximum deficits. The maximum deficits occur much further downstream (10 - 30 miles or 16 - 48 km) and water quality standards are violated much more frequently.

Further verification of the model can be achieved by simulating the stream response to hydrologic and waste inputs for, say, the year 1969. However, such a time-consuming task was not deemed justifiable in view of the accuracy already obtained and the study objectives.

Results

Based on NOAA records (Asheville, North Carolina), the total precipitation that fell over Des Moines, Iowa, during 1968 was 27.59 inches (701 mm). STORM computed a total runoff of 10.28 inches (261 mm) over a watershed area of 49,000 acres (19,600 ha), for an overall urban area runoff coefficient of 0.37. There were 65 days in the year during which rainfall was recorded, from which 58 wet-weather events were defined. The results are presented in the form of minimum DO frequency curves for the wet-weather and dry-weather periods throughout the calendar year. The reader is referred again to Table VII-5 for a review of the multitude of options modeled in the Des Moines application.

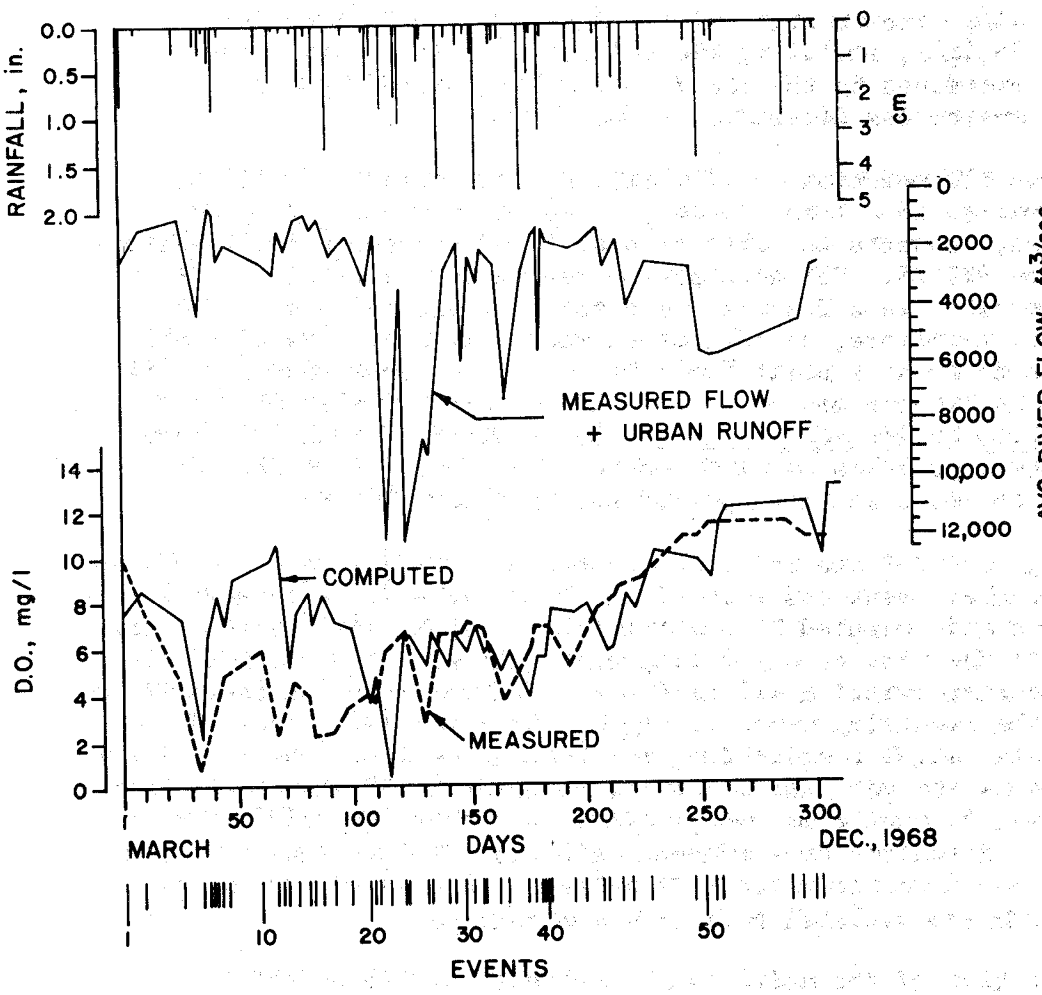

Figure VII-9. Application to Des Moines, Iowa. Measured and computed values of DO at 5.6 mi (9.0 km) downstream from confluence of Raccoon and Des Moines Rivers.

It is appropriate to examine first the model estimates of DO concentration in the Des Moines River for conditions assumed to exist in 1968 during periods of urban runoff:

1. combination M = 5, all waste inputs;

2. secondary treatment (85 percent BOD removal) of DWF, J = 2;

3. no stormwater treatment, L = 1;

4. river flow 100 percent of measured flow, K = 3, and

5. the fraction of combined area is 8.16 percent of the total urban area.

Figure VII-10, Minimum DO Frequency Curves for Existing Conditions in the Des Moines River, illustrates all waste inflow combinations. The curves indicate clearly that all combinations including a substantial amount of wet-weather flow (WWF) result in a drastic decrease in river minimum DO concentrations. For example, 42 percent of all the wet-weather events throughout the year produced conditions in the receiving water that caused minimum DO levels below 4.0 mg/1. Combined sewers contributed WWF from only 8 percent of the total urban area modeled, yet the BOD_c concentration was sufficiently high to inflict an appreciable reduction in DO levels when compared to DWF sources during periods of runoff.

Figures VII-11 through VII-15, Minimum DO Frequency Curves, present the results obtained by considering all waste inputs (combination M = 5, Table VII-5) while varying the other parameters. Figure VII-11 displays the minimum DO frequency curves obtained by varying the percent of the total urban area served by combined sewers. There is a substantial, but not drastic, decrease in water quality when the extreme conditions are compared: an area served only by separate sewers (0 percent combined) versus an area served exclusively by combined sewers. The curves support the theory that total separation of sewers is not the answer to the control of urban runoff pollution. The curves in this figure all represent secondary treatment of DWF, no urban runoff treatment, and full river flow.

Figure VII-12 shows the relative effect of urban stormwater runoff in the upstream portions of the drainage basin. As explained earlier, this effect is modeled by reducing upstream river flow to three different fractions of its actual measured value. DWF is given secondary treatment (85 percent BOD removal), while WWF is untreated. Thus, the only flow in the river consists of DWF and urban runoff when modeling discharge into a dry river bed (K = 1, Table VII-5). Variation of upstream river flow does not reveal large differences in receiving water quality, as might be expected, because of the relatively large volumes of stormwater runoff discharged by the urban area into the river: